design flood flows manual

Santa Clara Valley Water District

~~1. Mr Gill~~ DKG
2. File

memorandum

GK

TO
George Korbay/David Gill

FROM
John Richardson

SUBJECT
Coyote Creek Hydrology

DATE
October 20, 1977
1977

The attached table shows design flows for Coyote Creek from Anderson Dam to Highway 17.

Bill Brick of the Corps and Jim Schaaf of Nolte (representing the Federal Insurance Administration) have reviewed the resulting design flow values with their management and both parties agree to use the flow values as shown in their studies.

The Corps will use these flows in their feasibility study report but must reexamine the hydrology in the final design stage if and when the project is authorized for construction. The F.I.A. will use the flows as the 1 percent flood for their insurance study.

I recommend we adopt this table as the design flow for Coyote Creek. Attached is a draft letter to the Corps transmitting the design flow and giving our concurrence.

John

Head, Project Development Branch
Design and Construction

Attachments

cc: G. Watson
A. Saah
L. Preston
J. Richardson

JLR:amcd

Richardson/10/19/77

COYOTE CREEK DESIGN FLOWS

Reaches	With Los Osos Dam (cfs) (1)	Without Los Osos Dam (cfs)
Anderson Dam to Upstream of Fisher Crk	3000	15,000
Fisher Creek to Upstream of Upper Silver Creek	7900	15,000
Upper Silver Creek to Upstream of Lower Silver Creek	7400	14,500
Lower Silver Creek to Upstream of Upper Penitencia Creek	12,000	14,500
Upper Penitencia Creek to Highway 17	14,500	14,500

(1) Los Osos Dam is located above Coyote Reservoir and will regulate 109 square miles of watershed. The computation of the design flow reservoir assumes 30,000 A.F. of flood control storage in Los Osos.

Santa Clara Valley Water District

MEMORANDUM

TO: Distribution

FROM: Wendy Chang

SUBJECT: Updated Uvas Reservoir storage-discharge information - the main cause of increase in Uvas Creek design flows

DATE: August 5, 2004

In updating the South County hydrology, a newly updated Uvas Reservoir storage-discharge curve was applied to the HEC-1 hydrology model. The difference in storage-discharge rating between the 2004 and 1976 values is believed to be the main cause of the significant increase in design flows.

The updated storage-discharge curve is based on the most recent survey of the Uvas Reservoir Dam and Spillway which was completed in July 1996 by the Land Surveying and Mapping Unit.

Attached package contains a table comparing 2004 and 1976 flow rates, a graph comparing 2004 and 1976 storage-discharge curves, copy of COE's General Design Memorandum for Uvas Creek Levee at Gilroy and calculations of the updated Uvas Reservoir storage-discharge curve.

If you have further question, please call me (X2623).

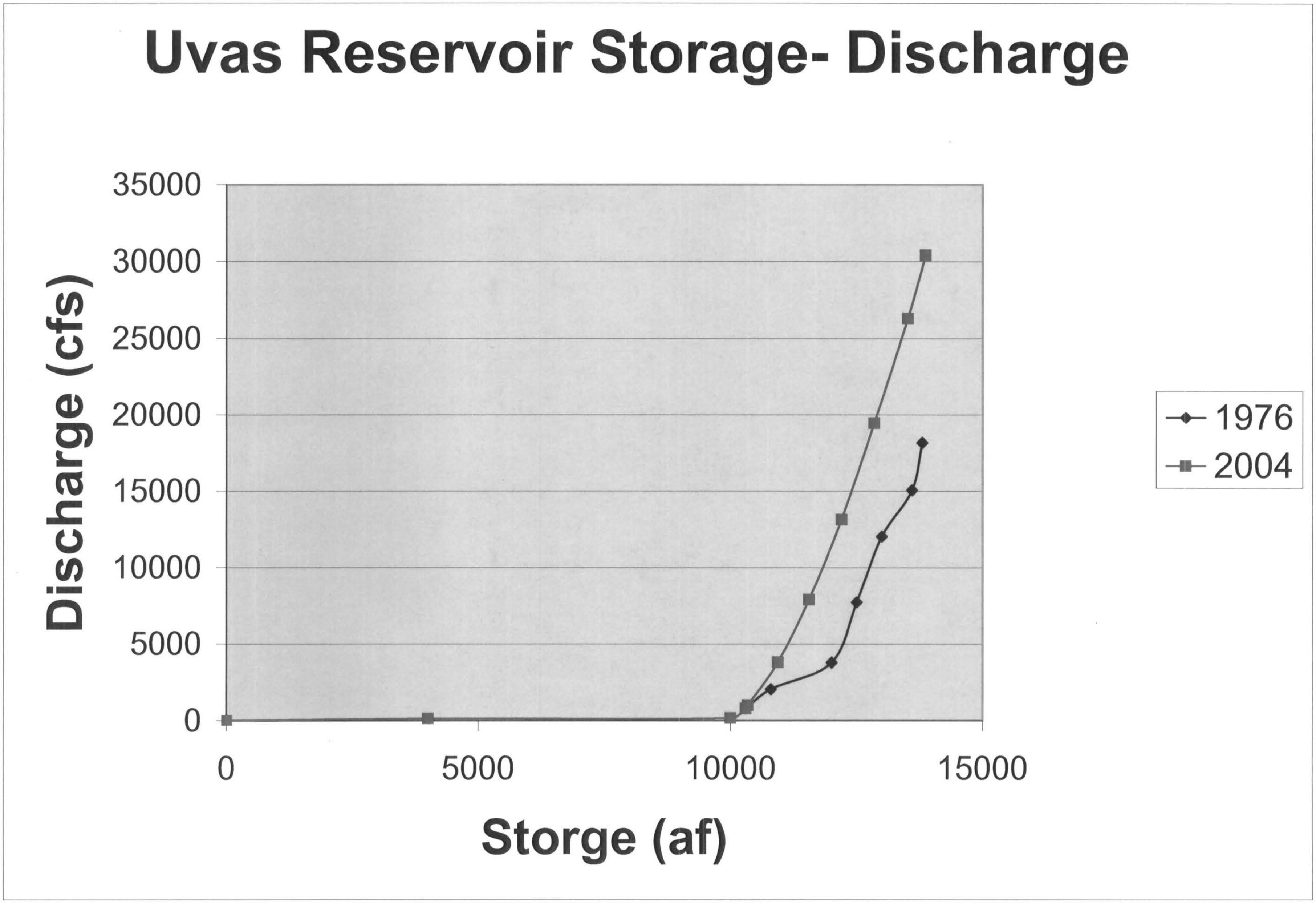
Attachment: (1) Table comparing 2004 and 1976 flow rates.
(2) Graph comparing 2004 and 1976 storage-discharge curves.
(3) Copy of COE's General Design Memorandum for Uvas Creek Levee at Gilroy
(4) Calculations of the updated Uvas Reservoir storage-discharge curve

CC/enc: Marc Klemencic, Nai Hsueh, Katherine Oven, Scott Katric, Jim Wang, Bal Ganjoo, Nahm Lee, Sue Tippets, Yvonne Arroyo

Memo uvas hydrology.doc

One Percent Design Flow Rates Comparison

Location	Drainage Area (sm)	2004 flow rate (cfs)	1976 flow rate (cfs)
Uvas d/s Reservoir Canyon Creek (near Thomas Road)	69	18200	14200
Uvas u/s Pajaro River	87	21200	16300


Uvas Reservoir Storage- Discharge
Discharge (cfs)
35000
30000
25000
20000
15000
10000
5000
0
0
5000
10000
15000
Storge (af)
1976
2004

NOVEMBER 1984

PAJARO RIVER, CALIFORNIA

Uvas Creek Levee at Gilroy

GENERAL DESIGN MEMORANDUM

US Army Corps of Engineers

Sacramento District

TC
425
P35.P35
1985

the left bank. This levee was constructed between 1937 and 1955. Present condition of this existing levee is marginal to poor.

1-04. Description of Authorized Plan. - The project as authorized in 1944 consists of channel improvements on the Lower Pajaro River at Watsonville and levee modifications on Uvas Creek (formerly Carnadero Creek) near Gilroy. The channel improvements at Watsonville were completed in 1949. The 1944 authorized plan of improvement for Uvas Creek consists of modification of the existing levee on the left bank from river mile 7.70 to river mile 9.25 (measured from the Pajaro River) and the construction of a short tie levee. This plan was to afford protection to the City of Gilroy from all floods up to the 100-year flood.

1-05. Description of Reformulated Plan. - The reformulated plan for the Uvas Creek levee is described in Appendix 1, "General Design Memorandum, Phase I." Briefly stated, this plan consists of a levee designed for SPF protection along the left side of Uvas Creek from approximately 2,000 feet downstream of Thomas Road to approximately 1,300 feet upstream of Miller Avenue. A new levee would be constructed from about 2,000 feet downstream of Thomas Road to about 1,100 feet downstream of Miller Avenue. About 1,100 feet of existing levee downstream of Miller Avenue would be reconstructed while approximately 1,300 feet of the recently constructed levee upstream of Miller Avenue would be raised. The plan includes the relocation of Thomas Road with the construction of a new bridge approximately 100 feet upstream of the existing crossing. Miller Avenue would be raised to match the project levee and several relatively minor utility relocations would be required. Recreation facilities consisting of an asphalt paved bike trail, graded earth hiking trail, and a parking and staging area are included in the plan. These changes to the 1944 authorized plan were presented in a Post Authorization Change Notification Report approved by OCE on 4 August 1982.

1-06. Requirements of Local Cooperation.

a. The authorized local cooperation requirements for the Pajaro River project at Gilroy are:

(1) provide without cost to the United States, all lands, easements, and rights-of-way necessary for the construction of the project;

(2) make at their expense all necessary changes in existing improvements, including utilities and highway and railway bridges;

(3) hold and save the United States free from damages resulting from construction of the works; and

(4) maintain and operate all works after completion in accordance with regulations prescribed by the Secretary of War.

b. A Post Authorization Change Notification Report approved by OCE in August 1982 modified the requirements of local cooperation. The changes in local cooperation requirements include 50 percent cost sharing of recreation features, application of the reimbursement clause in Section 3 of the 1936 Flood Control Act, and additional requirements as a result of subsequent Federal legislation for flood control projects.

direction. This coarser material will allow groundwater recharge, but seepage on the landside of the proposed levee will be negligible.

d. Settlement. - Consolidation settlement will vary proportionally with the height of the levee. A consolidation settlement of 4 inches will occur at the maximum design levee height of 10 feet. The levee will be overbuilt accordingly to compensate for expected settlement. It will be the local sponsor's responsibility to accomplish any additional work to keep the levee at or above the ultimate design levee grade.

e. Construction Requirements. - Grubbing and stripping will be required under all levee fill. Grubbing will include the removal of bushes, trees, roots and debris. Stripping will consist of the removal of organic topsoil to an average depth of 6 inches. The stripped surface will be proof-rolled prior to placing embankment materials. Fill material will be compacted to 95% standard density at optimum moisture content. Loose lift thickness will not exceed 8 inches. Slopes to receive riprap will be graded to 1V to 2H or flatter. Grubbing and stripping will be required prior to slope shaping.

3-03. Hydrology. - Detailed hydrologic studies and data were developed for the project and presented in "Hydrologic Engineering Office Report, Uvas-Carnadero Creek, Santa Clara County, California," dated May 1979, which was approved by South Pacific Division on 3 August 1979. Hydrologic data pertinent to the design of the project is summarized in Appendix 1, "General Design Memorandum, Phase I."

3-04. Hydraulic Design.

a. General. - The project design provides standard project flood (SPF) protection along the left bank of Uvas Creek through the City of Gilroy. In designing the project features, a concerted effort was made to utilize the alignment of existing levees. Channel excavation was minimized to protect existing riparian vegetation. Project channels were designed and analyzed with the guidance of EM 1110-2-1601, "Hydraulic Design of Flood Control Channels."

b. Project Design Flow. - The project channel was designed to safely pass the peak SPF discharge of 18,800 cfs.

c. Water Surface Profiles. - The current version of the HEC-2 Computer Program, "Water Surface Profiles" (Modification 54, Error Correction 05) was used to compute the project design water surface profile by the Standard Step Method. The HEC-2 data set, developed to simulate project conditions, was based on channel surveys taken in August 1983 for the reach between Stations 0+00 and 86+00 and on surveys taken in October 1976 for the reach upstream of Station 86+00. Throughout the entire project reach, the flow regime will be subcritical (tranquil) flow. Manning's equation was used to evaluate the effect of boundary roughness, i.e., friction losses, on water surface stages and velocities. Profiles were developed using maximum roughness values to set levee heights and minimum roughness values (25 percent reduction) to evaluate the need for slope protection and erosion potential.

(1) Minor losses used in the computations included transition (expansion and contraction) and bridge pier losses. Loss coefficients used for contraction and expansion were 0.1 and 0.3, respectively, while values of 0.3 and 0.5, respectively, were used for abrupt changes. Subcritical flow conditions, Class A low flow, prevail for all flows up to and including the project design flow; therefore, bridge losses were calculated with the Yarnell equation with a pier coefficient, K_s, of 0.9.

(2) As subcritical flow conditions prevail throughout the study reach, water surface computations began with known downstream hydraulic conditions and proceeded upstream. The design starting water surface elevation at the downstream end of the project levee was taken as elevation 202.5 feet NGVD based on water surface profiles computed for the reach downstream of the project.

d. Freeboard. - Freeboard was designed by establishing the project levee grade at least 2 feet above the SPF (18,800 cfs) water surface profile or at least 3 feet above the 100-year (17,000 cfs) water surface profile, whichever was greater. Since the SPF and 100-year water surface profiles differ by less than 1 foot throughout the project reach, the freeboard is always in excess of 2 feet. The difference between the SPF and 100-year water surface profiles in the reach between Station 14+00 and Station 27+00 is less than 1/2 foot. The project levee grade established utilizing this freeboard design is well above the energy grade line of the SPF throughout the project reach. Additional levee height of 1 foot above freeboard in the reach between Station 20+00 and Station 27+00 was added due to the potential for water surface superelevation through this reach. The freeboard design is considered adequate because the flood control project is a one-sided levee system. The project levee would be located on the left bank of the stream, allowing high flows to spread across the broad natural channel and the low-lying lands beyond the opposite channel bank. For this reason, erratic hydrologic phenomena, the accumulation of trash, debris and vegetal growth in the channel, and other unknown factors would not have a significant adverse impact on the levee system. If floods greater than the SPF should occur, the flow regime at Santa Teresa Boulevard bridge would change from open channel to pressure conditions, thus dampening the effect of the higher peak downstream. Some excess flows would be diverted down a swale along the right bank, and would spread across the low-lying lands beyond the right channel bank. This natural overflow would prevent weir flow across the bridge. In the event flow rates substantially in excess of the SPF should occur, water would continue to back up upstream of Santa Teresa Boulevard bridge and would flow around both the right and left ends of the bridge, outflanking the left bank levee downsteam. It is unlikely that the bridge would fail, as it is a reinforced concrete structure recently constructed by Santa Clara County. Another consideration in establishing the freeboard design is that the project levee would have a somewhat wider crown (18 feet) in order to support the local sponsor's maintenance equipment (see paragraph 4-02.). The 18-foot crown width would provide added protection and would allow good access in the event that a flood fight is required during a flood event. To the extent required, the risk associated with overtopping of the project levee has been eliminated for the design flood and minimized for floods in excess of the design flood.

e. Bank Protection. - Project features were designed to preclude the necessitiy for extensive stone protection. The project levee is set back from the main channel in lower velocity overbank flow areas to minimize

PAJARO RIVER BASIN
UVAS - CARNADERO CREEK

SANTA CLARA COUNTY, CALIFORNIA

GENERAL DESIGN MEMORANDUM

PHASE I

MAIN REPORT AND ENVIRONMENTAL STATEMENT

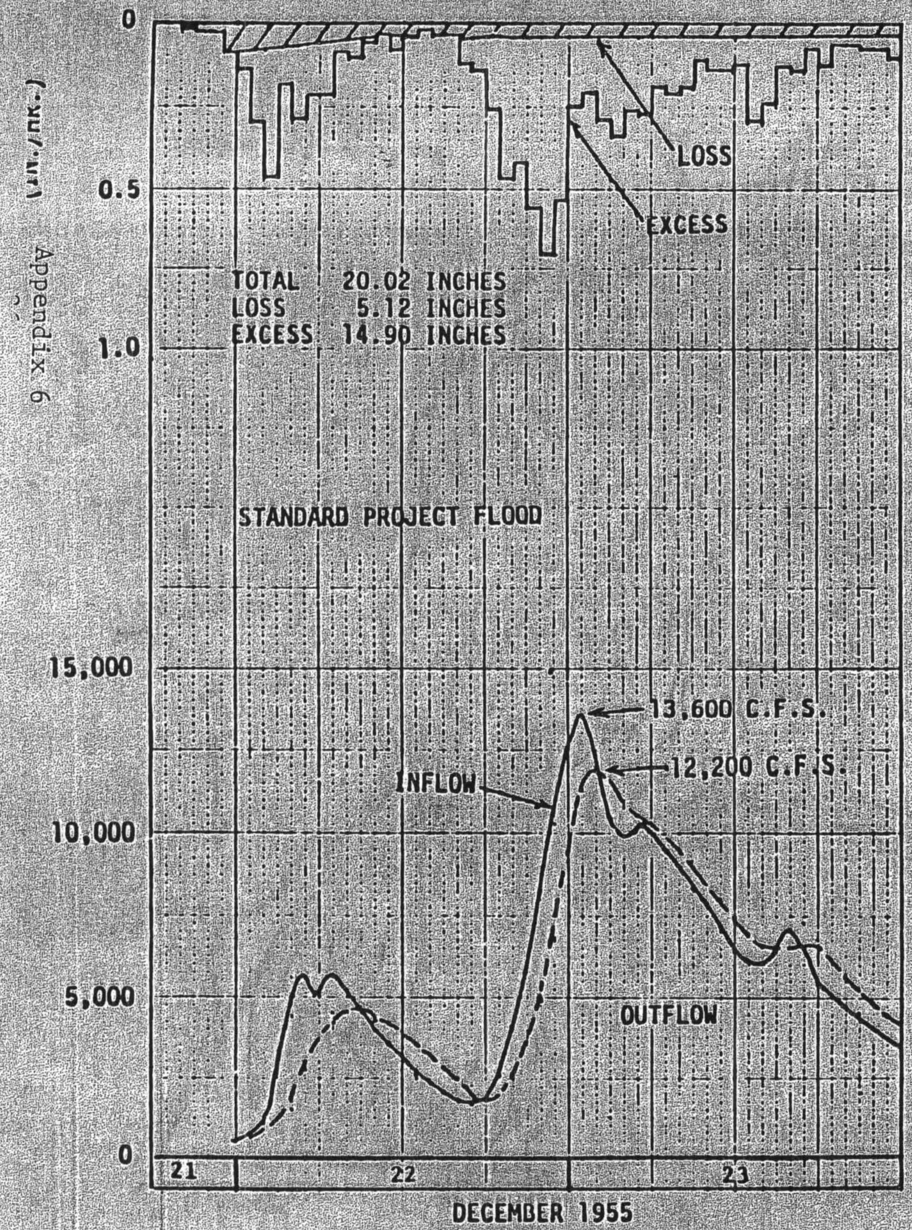
JULY 1981

DEPARTMENT OF THE ARMY
SAN FRANCISCO DISTRICT, CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

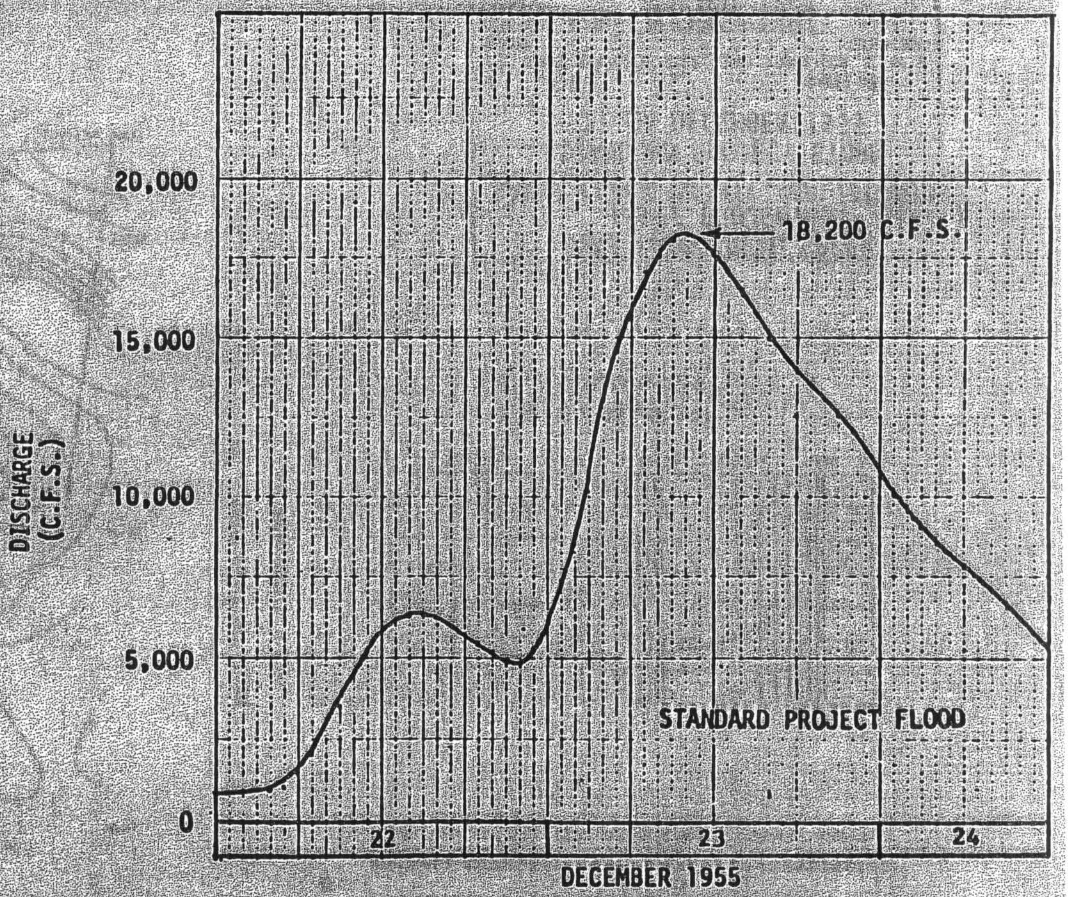
2.2.301-1

Appendix 6

Appendix 6



UVAS CREEK AT UVAS RESERVOIR
DRAINAGE AREA 30.4 SQ. MI.



UVAS CREEK AT THOMAS ROAD
DRAINAGE AREA 68.9 SQ.MI.

UVAS-CARNADERO CREEK
LEVEE PROJECT
SANTA CLARA COUNTY, CALIFORNIA

STANDARD PROJECT FLOOD
HYDROGRAPHS

IN. SHEET SHEET NO
U.S. ARMY ENGINEER DIST. SAN FRANCISCO. C OF E
DRAWN: FILE NO
TRACED: TO ACCOMPANY REPORT
CHECKED: AEM DATED MAY 1979

PLATE 11

Checked w/ Nahm
12/19/03

STOR-Q (Revised)

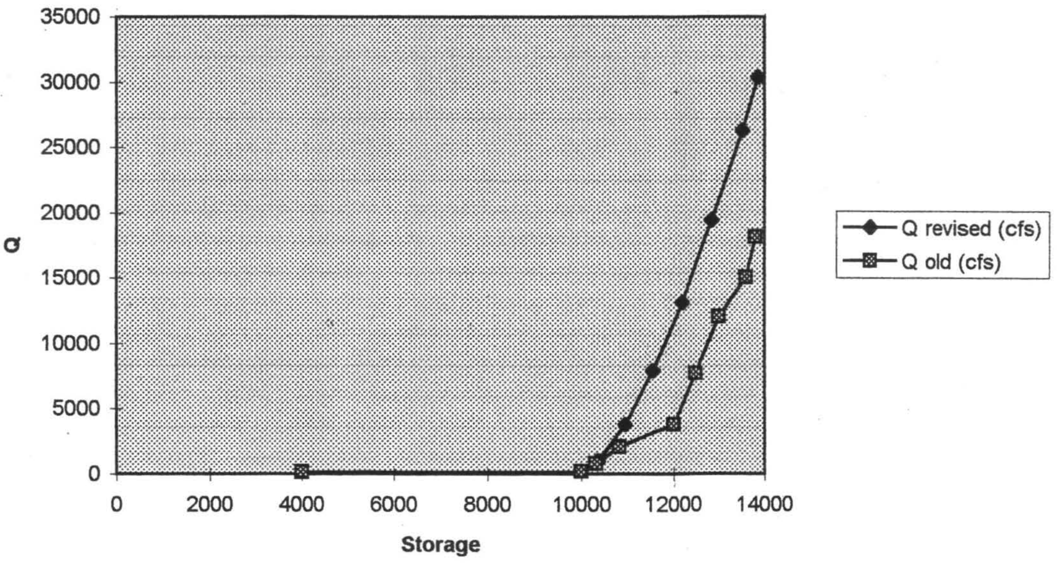
STORAGE DISCHARGE RELATIONSHIP FOR UVAS DAM
BY W. CHANG (OCT. 96)

L (ft)	P (ft)	Ho (ft)	P/Ho	Co 1)	Co (corr) 2)
171.39	5.5	11	0.50	3.8	3.76

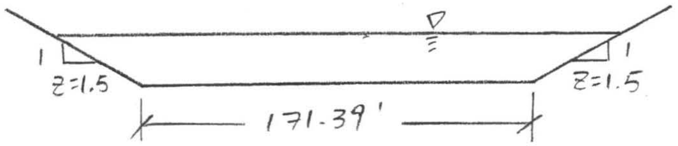
ELEV (ft)	He (ft)	He/Ho	3) C/Co (corr)	C	hd+d/He	2) Cs/C	4) C (corr)	5) Q ogee (cfs)	6) Q v-notch (cfs)	7) STORAGE (AF)	Q revised (cfs)
487.5	0	0								9896	
489	1.5	0.14	0.84	3.16	4.87	1	3.16	994	10	10340	1005
491	3.5	0.32	0.88	3.31	2.66	1	3.31	3713	86	10935	3799
493	5.5	0.50	0.92	3.46	2.05	1	3.46	7647	266	11555	7913
495	7.5	0.68	0.95	3.57.	1.77	1	3.57	12574	578	12200	13152
497	9.5	0.86	0.98	3.68	1.61	1	3.67	18400	1043	12845 12522	19443 16497
499	11.5	1.05	1	3.76	1.50	0.98	3.68	24629	1682	13516	26311
500	12.5	1.14	1.02	3.84	1.46	0.98	3.74	28323	2072	13865	30395

1) EFFECT OF DEPTH OF APPROACH ON COEFFICIENT, FROM FIG. 9.23 "DESIGN OF SMALL DAM"
2) EFFECT OF D/S APRON INTERFERENCE AND D/S SUBMERGENCE ON COEFFICIENT, FROM FIG. 9.27 "DESIGN OF SMALL DAM"
3) EFFECT OF HEADS DIFFERING FROM DESIGN HEAD ON COEFFICIENT, FROM FIG. 9.24 "DESIGN OF SMALL DAM"
4) C must be less than 3.8
5) Q ogee = CL(He^3/2)
6) Q v-notch = 2.5*slope*(He^2.5)
7) DATA FROM SCVWD RESERVOIR STORAGE AND SURFACE AREA RATING CURVES, MAR. 91

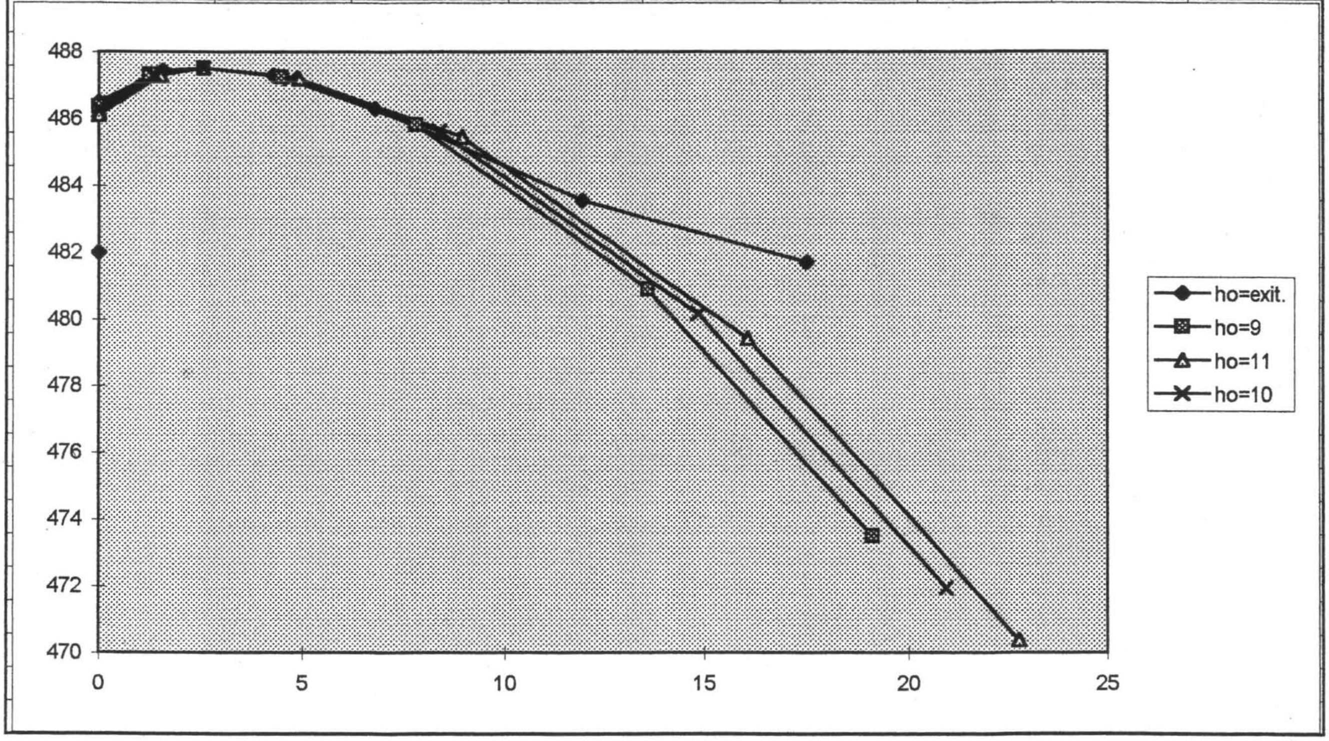
STORAGE (AF)	Q old (cfs)
4000	135
10000	175
10300	805
10800	2065
12000	3810
12500	7745
13000	12040
13600	15065
13800	18180



There are some calculation errors in the old spillway rating curve. Please referred to the study folder for further details.



	DIST	ELEV				
		ho=exit.				
TOE OF OGEE	0	482				
	0	486.5				
	1.55	487.45				
TOP OF OGEE	2.55	487.51				
	4.3	487.3				
	6.8	486.3				
	11.96	483.55				
	17.5	481.71				
ho		9		11		10
	DIST	ELEV	DIST	ELEV	DIST	ELEV
		ho=9		ho=11		ho=10
	0	486.367	0	486.113	0	486.24
	1.233	487.321	1.507	487.279	1.37	487.3
TOP OF OGEE	2.55	487.51	2.55	487.51	2.55	487.51
	4.503	487.249	4.937	487.191	4.72	487.22
	7.797	485.827	8.963	485.453	8.38	485.64
	13.62	480.904	16.08	479.436	14.85	480.17
	19.11	473.506	22.79	470.394	20.95	471.95



Calculate Approach Channel Losses for Design Head and Co

Assume no ~~velocity head~~ approach velocity (wide approach channel)

$\therefore h_a = 0$

$H_o = 11$ ft (see fitting the O.G.)

$\frac{P}{H_o} = \frac{5.5}{11} = 0.5 \Rightarrow \boxed{C_o = 3.8}$ (Fig. 9-23)

Effect of D/S Apron

$\frac{(h_d + d)}{H_e} = \frac{11 + 5.8}{11} = 1.53$

From Fig. 9-26 for $\frac{h_d + d}{H_e} = 1.53$

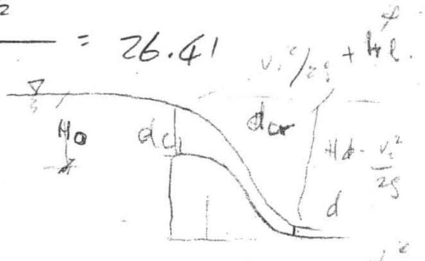
$\frac{h_d}{H_e}$ @ supercritical flow $= 1.05$

if supercritical flow prevails $h_d = 1.05 \times H_e$
$= 1.05 \times 11 = 11.55$

$h_d + d = 16.8 \Rightarrow d = 16.8 - 11.55 = 5.25$

D/S velocity $= \frac{q}{d} = \frac{C h_o^{\frac{3}{2}}}{d} = \frac{3.8(11)^{\frac{3}{2}}}{5.25} = 26.41$

$h_v = \frac{(26.41)^2}{64.4} = 10.83$



$\frac{h_v}{H_e} = \frac{10.83}{11} = 0.984$ and $\frac{h_d + d}{H_e} = 1.53$

$\frac{v_{cr}^2}{2g} + d_{cr} = \frac{v_1^2}{2g} + d$

From Fig. 9-26 verified that flow is supercritical $1 = \frac{v_{cr}}{\sqrt{2g}}$

therefore it can be assumed that D/S effect is caused by apron influence only

$\frac{h_d + d}{H_e} = 1.53 \Rightarrow \frac{C_s}{C_o} = 0.989$

$\therefore C_s = 3.8 \times 0.989$

$\boxed{C_s = 3.76}$

TIDES

IN SANTA CLARA COUNTRY

by Santa Clara Valley Water District

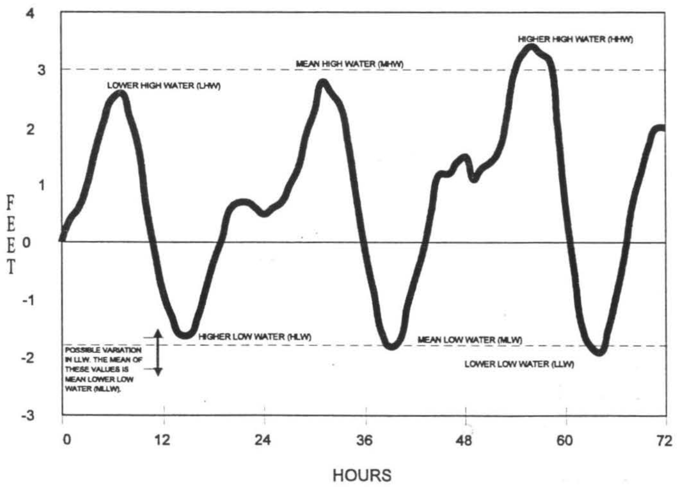
INTRODUCTION

Tides have always been of interest to sailors, clam diggers, and others, but the Santa Clara Valley Water District (District) also has need for special knowledge of tides in the south San Francisco Bay. Since all creeks and flood control channels eventually discharge into the South Bay, it is important to know the magnitude of possible extreme tides for proper design of levee heights, some bridges, and other structures. Other unique flood control facilities such as the Palo Alto Flood Basin required special knowledge of the tides because it temporarily stores storm water until the tides recede and the water is released into the Bay. Construction activities, wetlands mitigation design and operation, and water quality testing also require knowledge of tidal action.

TIDAL CYCLE

A typical daily cycle is shown in Figure 1. There are two high tides (higher high water and lower high water) and two low tides (higher low water and lower low water) in 1 day. The average of a series of lower low water data is called mean lower low water (MLLW) and that of higher high water data is called mean higher high water (MHHW). Both MLLW and MHHW vary with geography and are specific to individual locations (Figure 2).

Figure 1—Typical Tidal Cycle



TIDE TABLES

Tide tables used for the South Bay estimate the height of the tide relative to MLLW at the National Ocean Service stations at Golden Gate, which is 2.86 feet below National Geodetic Vertical Datum (NGVD, commonly referred to as mean seal level datum). MLLW at the District's Gold Street tide gage in Alviso is 4.5 feet below NGVD. The tide tables include an adjustment factor to translate the tide information for the Golden Gate to the Alviso location. For high tides, the adjustment factor for Alviso is +3.3 feet, which should be added to the tide table value. The resulting value is the tide height above MLLW at Alviso, but not the height above NGVD or MLLW at the Golden Gate. For example, a prediction of a 3.0 foot tide at the the Golden Gate would mean that the tide at Alviso is 6.3 feet (3.0 + 3.3 feet adjustment factor) above MLLW at Alviso. MLLW at Alviso is 4.5 feet below NGVD (Figure 2) and so the tide at Alviso is 1.8 feet above NGVD (-4.5 + 6.3 feet). Figure 3 is the result of a tide study that translates the high tide elevations at Golden Gate to Alviso. The tide tables also indicate that high tides at Alviso occur 1.2 hours (72 minutes) after tides at the Golden Gate.

Figure 2

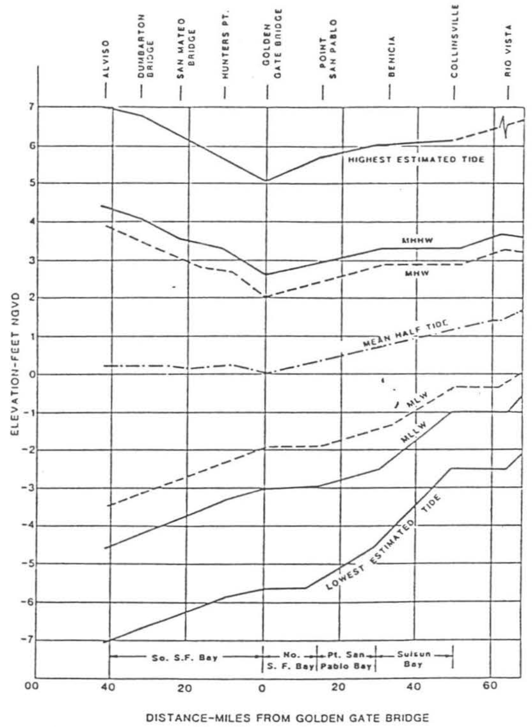


Figure 3—Relationship Between the High Tide Elevation Elevations at Golden Gate and Alviso

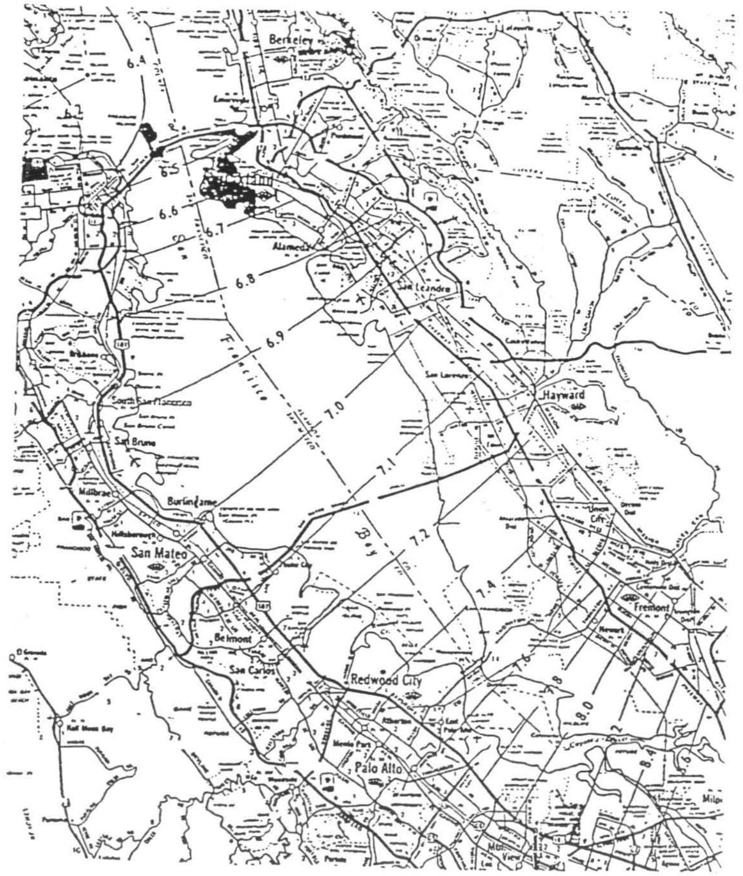
Golden Gate		
Tide Tables (Above MLLW)	Above NGVD	Gold Street in Alviso Above NGVD
5.00	2.14	4.10
5.50	2.64	4.68
6.00	3.14	5.26
6.50	3,64	5.84
7.00	4.14	6.42

More than 90 percent of the actual observed tides are higher than the predicted tides with 3 percent more than a foot higher. The National Ocean Service prepares the tide tables based on astronomic conditions, which account for only 70 percent of the influence on tidal action. Other factors are meteorological condtions such as barometric pressure or wind speed and direction. A drop of 0.1 inch in barometric pressure will increase the tide height by 0.1 foot. The highest tide recorded at Golden Gate since data was first collected in 1854, was 9.1 feet above MLLW on January 27, 1983, when the tide tables predicted only 7.1 feet above MLLW. An example of the possible variation in high tide was February 16, 1980, when the tide tables predicted a tide elevation of 6.6 feet (MLLW) at Golden Gate but the observed tide was 8.0 feet above the MLLW (5.14 feet NGVD). The observed tide at Gold Street was 6.86 feet NGVD, 1.72 feet higher than at Golden Gate..

EXTREME TIDAL EVENTS

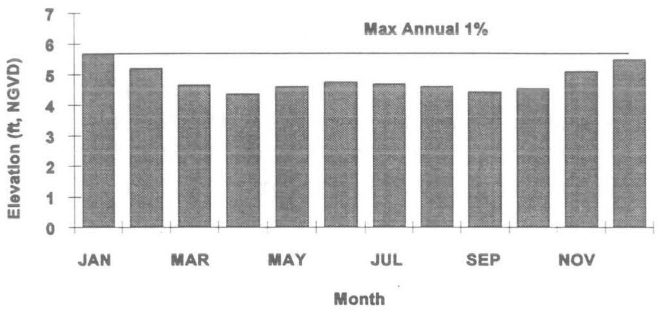
As a result of the high tides in 1983, the U.S. Army Corps of Engineers analyzed the maximum tide in each year to compute the 1 percent or 100-year tide (a tide height that has a 1 percent chance of occurring in any year). Figure 4 shows the 1 percent tide elevations as they increase toward the South Bay estuaries. The estimated 1 percent tide in Santa Clara County ranges from 7.6 to almost 9.0.

Figure 4—Adopted 100-Year Tide Elevations



A similar analysis was done using the maximum annual tide in each month. The expected 1 percent tide for each month is shown in Figure 5. The 1 percent tides in the months of November through February are nearly 1 foot higher than that of March through October.

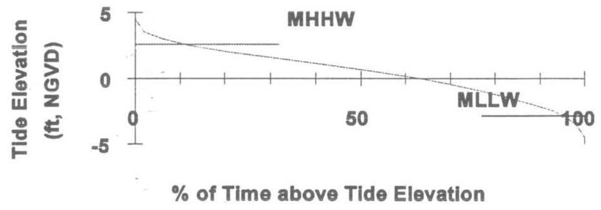
Figure 5—Variations of Estimated 1 Percent Tide Elevation by Month, Tide Elevation at Golden Gate Tide Gage for Each Month from 1898 to 1991



TIDAL CYCLE DURATION

Often it is necessary to know how long the tides could be expected to inundate different baylands elevations. The tide data at the Gold Street gage was used to compute the percent of time the tide equalled or exceeded a specific elevation. Figure 6 shows the percentage of time the tide is above each elevation.

Figure 6—Duration of Tide Elevation at Gold Street in Alviso



TIDE ELEVATION MAP FOR STREAMS

Figure 7 shows the extent to which streams are affected by 4 to 7 feet tide elevations and MHHW. The rise and fall of stream stage observed in other lengths of the creeks is a function of stream response to rainfall, not tides.

TIDE LEVELS EFFECT ON DIFFERENT STREAM FLOW

Figure 8 shows the effect of tide levels on streams during summer. Since the flow in the streams is low to zero, both high and low tides would push water up into the stream and raise the water level. The varied water levels are the result of the tide cycle only. However, when streams have a much higher inflow, the water levels are not raised with a high tide because their elevations are higher than the tide. Figure 9 shows that the tide would raise the water level only when the tide elevation is above the stream stage.

Fig. 8 Effect of Tide Levels on Low Stream Flow

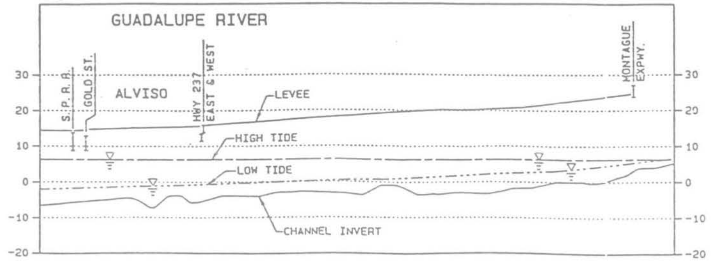
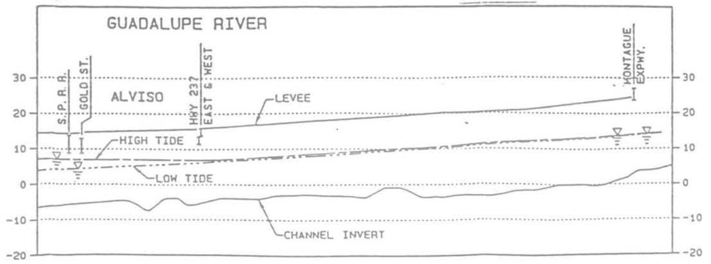


Fig. 9 Effect of Tide Levels on Moderate Stream Flow



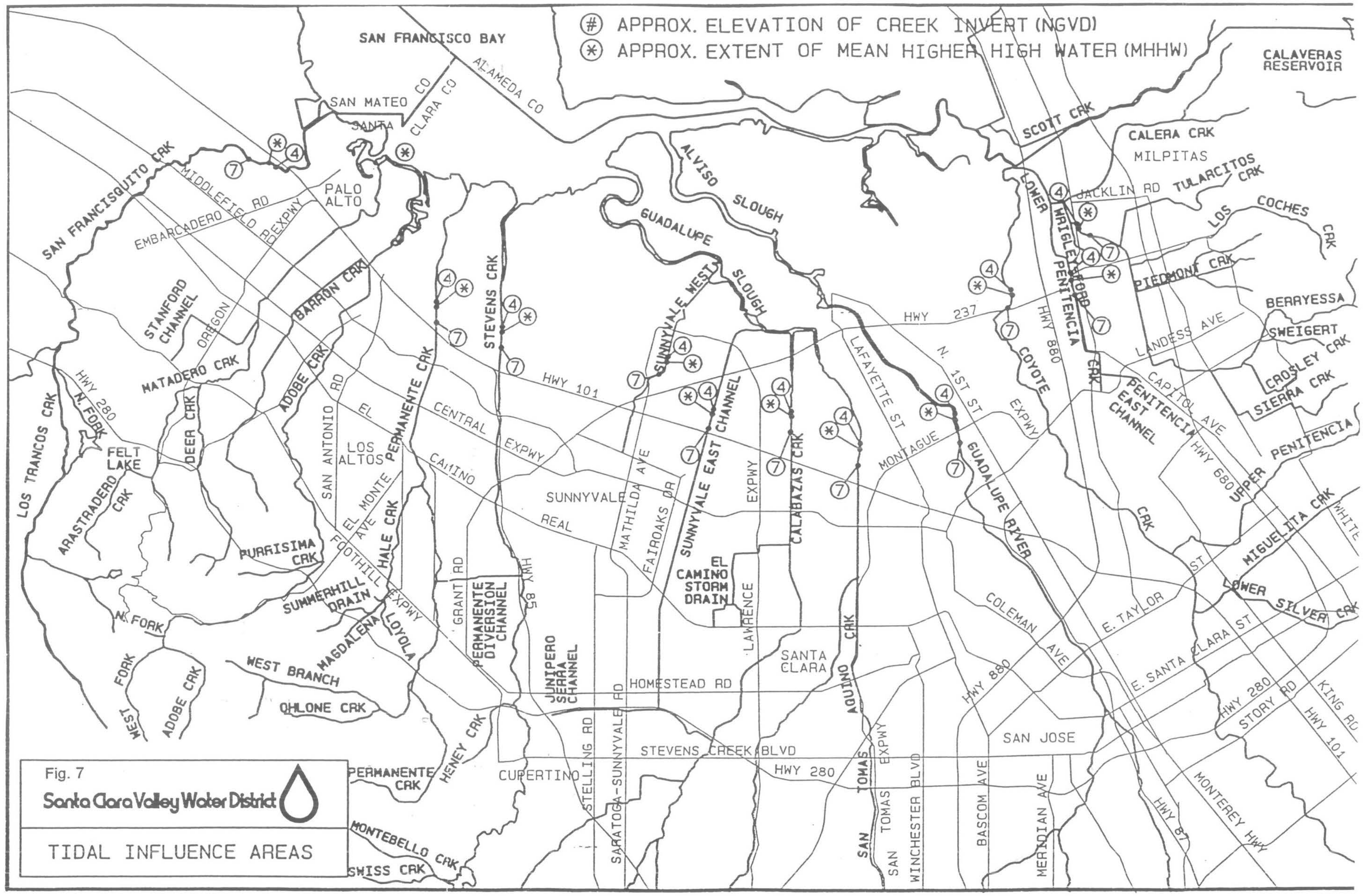


Fig. 7
Santa Clara Valley Water District
TIDAL INFLUENCE AREAS

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COUNTY HYDROLOGY, REGIONAL APPROACH

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ABSTRACT: The effort to update the regional hydrology methods of the Santa Clara Valley Water District (District) entails a series of tasks that are long and difficult. This paper enumerates the tasks and provides a quick look at the problems and the progress that has been achieved towards finding solutions. The hydrology methods cover all types of watersheds, small, large, urban, rural, gaged, ungaged, regulated and unregulated. This is done to provide consistent design flow values for storm drains and major flood control facilities, for small detention ponds as well as major multipurpose reservoirs.

The first part of this paper addresses project management, funding, the difficulties encountered, and the back-up plans.

The second part of the paper addresses the gathering and qualification of basic hydrological raw data, the extraction of maximum annuals from that data, and the development of mathematical and statistical tools for the analysis of the resulting time series. In addition, the paper addresses the use of HEC-1 to develop hydrographs from rural and urban areas. For urban areas, an extra effort was made to develop an urban hydrology technique. This technique is used to evaluate the impact of increased imperviousness and channelization, due to various types of land uses, on the hydrologic responses of watersheds.

Lastly, this paper addresses the design criteria for detention ponds and their use as multipurpose facilities for flood and non-point source (NPS) pollution control.

INTRODUCTION

The issue of what area to include in a hydrologic study and how to provide for that inclusion can quickly become a political rather than a scientific problem. As a result of the cutback in State and Federal funds for programs in the field of hydrology and basic data gathering, local agencies like the District find that they must carry that regional hydrologic/hydrographic data collection burden. Currently, this is done by various agencies in a fragmented and uncoordinated way.

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To proceed with this hydrologic study, the Hydrology Section of the District began gathering basic streamflow and precipitation data from the immediate hydrometeorologic region that includes eight counties in the San Francisco and Monterey Bay Areas. Local, State and Federal (USGS and National Weather service) data were compiled for further qualification and manipulation.

As the study progressed beyond the data gathering stage, it became apparent that, with a little more effort, this localized hydrology study could be expanded to become a regional hydrology manual for the eight-county area at a relatively low cost of about $20,000 per benefitting agency per year for two years. Several meetings with representatives of these agencies were convened to discuss the major premise of the project and to identify the main goal. The approval and acceptance of this idea by all agencies was considerable. As a result, cost sharing agreements, layout plans and work elements were defined and every agency was invited to share in this effort. However, to date, while no agency has refused to participate, no agency has signed the cost sharing agreement. Consequently, the project has been scaled back to its original scope that is more narrowly defined for the District's Santa Clara County flood control programs.

This outcome is understandable for two reasons: 1) all local agencies are caught in a very tight budgetary squeeze, and 2) most agencies are involved in localized studies. I would note that the cost of these localized studies can be many times more than the individual cost of the proposed regional study.

BASIC STREAMFLOW DATA QUALIFICATION

As a result of the data qualification process mentioned above, it was found that USGS data did not correlate well with the District data; the format and time span of DWR data were different than the National Weather Service and District data.

These and other problems required that the data be checked for accuracy, consistency and homogeneity. A cursory check of the District's data indicated that the data were processed without the proper application of time and stage shifts of rating curves. Such errors resulted in data that were, at times, 100% off of the actual values. Rather than neglecting the issue, the District opted to correct and qualify its 1300 station-years of data spanning a period of approximately fifty years. Using current computerized hydrographic techniques and various USGS guidelines, the qualified data include, among other things, the proper application of shifts to the rating tables. This has made the data reliable and compatible with the advanced hydrologic and mathematical techniques that will be employed in the analysis.

ANALYSIS OF PRECIPITATION DATA.

Three hundred (300) precipitation stations with a total of approximately 15000 station-years of precipitation data were analyzed. Long term stations were correlated with short term stations, and the statistics of short term stations were positively adjusted. Improved statistics (means and standard deviations) were obtained and used in drawing a Mean Annual Precipitation map for the eight counties. An open file report for this work was prepared by the District.

In addition, the depth-duration-frequency analyses, done by DWR and Jim Goodridge, were updated and the final frequency regimes for all stations together with the corresponding intensities were calculated.

DURATIONAL ANALYSIS AND DATA EXPANSION

Durational Analysis programs (DURANAL) for streamflow, precipitation and tide data were written to interrogate huge amounts of basic continuous raw data that are qualified and stored in the Corps of Engineers Data Base HECDSS. These programs extract the maximum annuals of peaks and volumes of any desired duration. They can read bar as well as point data that are either continuous, hourly, daily or other type of data.

The logic of the DURANAL programs is based on defining the continuous raw data in terms of valleys and peaks with definite starting and ending points. The maximum annuals of various durations are obtained from these peaks and valleys. It requires minutes to run a station-year of data, rather than the hours that are needed for the older brute force programs that do the same work.

Approximately 75 streamflow and over 100 precipitation stations were processed using DURANAL. The resultant maximum annual time series were expanded and missing values were filled-in using DEXFIL mathematical models.

The DEXFIL computer program and its user's manual were written by Prof. Shen and Dr. Tabios both of U.C. Berkeley, in cooperation with the District. In summary, DEXFIL is based on a modular set of computer programs written in FORTRAN 77. These modules address the model identification, parameter estimation and model implementation. From a package of linear, multiplicative, exponential and threshold models, the program DEXFIL chooses the best model to expand and fill-in missing data based on a predefined ranking procedure. This ranking procedure is based on a series of normality and independence hypothetical testing of residuals.

FREQUENCY ANALYSES AND THE USE OF WRC BULLETIN 17B.

Using the qualified, continuous and homogeneous time series from above, improved statistics were calculated at various gaging locations (watersheds). This is done through the application of a revisited Water Resources Council Bulletin 17B (a streamflow frequency analysis bulletin) and an expanded Corps of Engineers

HECWRC program. The expansion of HECWRC was done to include the "slippage" alternative hypothesis for testing of outliers, where a certain number of observations have different distribution parameters than the rest of the observations. In addition, HECWRC was expanded to include the indirect method of moments and the method of maximum likelihood. The results were compiled in a computer program LOGP3D and a user's manual both written by Professor Shen and Dr. Tabios of Berkeley, with the cooperation of the District.

REGRESSION OF STATISTICAL PARAMETERS.

At gaged locations, the watershed parameters (area, mean annual precipitation, length and slope of creeks, etc,) were regressed, using the DEXFIL program, against the improved statistics of data from the corresponding gaging stations to obtain the regional regression equations. These equations are then used to define the statistical parameters at ungaged locations. The resultant regression equations cover a range of rural areas varying in size from 0.2 acres to as large as hundreds of square miles. This was done through some adjustment of the regression constants.

THE USE OF CORPS OF ENGINEERS PROGRAM HEC-1

HEC-1 is a very well documented and widely used public domain computer program that is based on the unit hydrograph and the kinematic wave theories. It contains a package of hydrologic tools that enable the user to generate runoff hydrographs (peaks and volumes) from precipitation. The frequencies of the peaks and volumes of the resultant hydrographs are assumed to be equal to the frequencies of input storms. Because this is not wholly acceptable, calibration, reconstitution and or balancing of these peaks and volumes against recorded or regional values (as obtained above) is required.

This premise is the major driving force behind the generation of regional equations through which the information about the frequency regimes of peaks and volumes of flood flows at all locations could be easily obtained with a measurable confidence. More-over, the use of such regional equations renders the use of all methods (the SCS, kinematic wave or even the rational method) compatible.

A report addressing the generation of regression equations is being prepared by the District. Because most of the data used in the regression analyses are obtained from rural areas, the above-mentioned equations are applicable only to rural areas where the base condition of any hydrologic analysis is defined. For urban areas the following procedures are applied to adjust the above-mentioned rural base condition.

URBAN HYDROLOGY

A typical valley floor urban drainage area in Santa Clara Valley is assumed to behave like a reservoir. The inflow to that reservoir is mainly from the rain falling over the urban area. The outflow from the reservoir is through the storm drains, if the creeks are leveed, or through the storm drains and the overland flow from the side streets or other adjacent areas, if the creek is not leveed.

Based on the general plan and the land use of every city/county, each urban area in this study is schematically divided into two separate sub-areas; the pervious sub-area and the impervious sub-area. Each of these two sub-areas has different characteristics: different time of concentrations and different loss rates, different routing functions, different curve numbers and in summary different hydrologic features. As a result of an extensive optimization of rainfall against the respective runoff from urban areas, sets of hydrograph parameters and watershed characteristics are obtained and regressed against each other. The resultant regression formulas (that include the ratio of imperviousness as an independent variable) are then used to generate unit hydrographs and other HEC-1 input parameters. Consequently, separate hydrographs from the pervious and impervious parts of urban sub-areas are obtained. These hydrographs are then combined and routed through the storm drain system, as the condition may require, to generate the design outflow from the urban watershed. Generalized guidelines were developed for routing through the storm drains. A report (Hydrology Of Urbanized Watersheds) that addresses this urban study was prepared by the District.

REGULATED WATERSHEDS

In watersheds regulated by reservoirs, the balanced inflow hydrographs are routed through the reservoirs using an appropriate storage-discharge relationship. Extra care was placed on the identification of the initial storage for these reservoirs. The initial storage of a reservoir that is coincidental with a probabilistic flood was determined by an adequate solution of the conditional probability problem between the incoming flood and the corresponding reservoir storage.

MULTIPURPOSE DETENTION FACILITIES.

Using the HEC-1 program and the above mentioned procedures, inflow hydrographs to the detention facilities are obtained. These hydrographs are then routed through detention facilities with acceptable hydraulic design features. The final design of any detention facilities (including the sizing of the storage, the outlet works and the spillway) is based on total watershed considerations. This means that the outflow from any detention facility is synchronized with the outflow from other areas to reduce the inadvertent impact of many detention facilities functioning erroneously to increase, rather than decrease, the induced flooding.

Operation rules and guidelines are particularly important for facilities that include non-point source pollution (NPS) control features. In such cases the regional solution of the pollution problems is done by operating the regional flood control detention facilities as multipurpose facilities. An important factor in the design of such facilities is the cost of the cleaning program. Hydrologists and engineers should be careful about the use of the "Design Storm" as defined by the "Storm Water Quality Control Report" as a design guideline. Such a storm is too small and has no duration and probability of occurrence to be of any practical value.

CONCLUSION

The application of the above hydrology methods results in consistent design flows for all kinds of watersheds that include all kinds of conditions. The method is based on qualified and updated data that are manipulated and analyzed using state-of-the-art programs. The use of the regional regression equations to define design peaks and volume of floods is essential in the development of hydrographs as specified in HEC-1 program.

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W F Carlsen

Distribution   John L. Richardson

Design Flood Flows Manual   January 11, 1979

Attached for your use is a copy of Design Flood Flows Manual. This Manual supersedes the old Manual of Design Flood Flows for Santa Clara County, dated April 1971, which should be returned to this Branch immediately.

We expect no major review and revision of our hydrology methodology for at least five years. Some flow values may be updated and changed and, also, flow values will be added as the need arises. At such time new dated sheets will be issued by the Project Development Branch to replace the old ones. For this reason, and in order to account for all copies, please retain the Manual in your possession and do not reproduce individual sheets.

Mr. Saah of this Branch is in charge of updating and maintaining this Manual. If you have any questions or need more information, please contact him.

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Head, Project Development Branch
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JAN 1979

SANTA CLARA VALLEY WATER DISTRICT

MANUAL OF DESIGN FLOOD FLOWS
FOR
FLOOD CONTROL FACILITIES
IN
SANTA CLARA COUNTY

This Manual Not to be Duplicated
or Removed from Office

November 1978

JAN 1979

Preface

This manual supersedes the "Manual of Design Flood Flows for Santa Clara County Flood Control and Water District" dated April 1971.

The design values printed herein should replace all other previous values for design of Flood Control Facilities and other flooding-related problems.

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Methods and Procedures

JAN 1979

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JAN 1979

SUMMARY OF METHODS AND PROCEDURES
TO DETERMINE DESIGN FLOOD FLOWS
FOR
FLOOD PROTECTION FACILITIES IN SANTA CLARA COUNTY

General

The District has established its flood protection criterion for watershed areas greater than four square miles based on flows which have a 1% chance of being equaled or exceeded in any one year. For watershed areas between one and four square miles the criterion is the 2% flood, and for watershed areas between 360 acres and one square mile it is the 4% flood.

The accompanying maps and tables represent the 1977-78 update of the design flood flows for use in the design of flood control facilities in Santa Clara County. They present the 1% and 10% design flood flows. Other design values (2% and 4%) are not presented herein; they could be obtained upon request. These values were calculated using the District's methods and procedures as augmented and modified by the recommendations of the panel of expert hydrologists* convened to review the methods in 1976. It was assumed in the derivation of these design flows that the channel system is free from major constrictions or significant flood storage (reservoirs are handled separately) and that the floodwaters at the design level are completely contained within channel banks or relatively narrow floodway limits.

The District's method of determining the design peaks and volumes of floods utilized all available sources of information to derive the most accurate values possible. In general, this method began with the use of Regional Regression Equations to compute peaks and volumes. These peaks and volumes were compared with the results of an analysis of reliable recorded streamflow data when such exist. The values were also compared with the results of a rainfall-runoff model before the final "design flow" classification is given. Where design flows are presently affected by reservoirs or by urbanization, special analyses accounting for their effects were performed. Finally, the design flows were compared with the results of other sudies and with previously determined design flows. The results of such comparison are useful to further define the design values.

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*The panel members were Leo Beard, Dave Dawdy, Neil Grigg, Douglas L. James, and Vujica Yevjevich. Their report, "Review of Basic Hydrology Methodology for Flood Control", Santa Clara Valley Water District, 1976, is available from the District library.

Hydrographs were derived utilizing the Corps of Engineers' standard project storm and the Clark Synthetic Unit Hydrograph technique. The peaks and volumes of these hydrographs were then adjusted to agree with those values derived from regional or local statistics. The flood hydrographs obtained this way are considered representative for use in the design of most flood control facilities. However, where flood traps (small volume peak skimmers) are being considered, a sensitivity analysis of the design relative to hydrograph shape should be conducted.

Following is a more detailed discussion of these methods. Full documentation of the analysis of each stream is available in separate reports.

Regional Regression Analysis

The backbone of the District's hydrology is the application of the Regional Regression Equations (1976) for the determination of flood peaks and volumes. These equations were obtained by analyzing recorded streamflow data (peak flows and their respective volumes) from stations within and outside the District. The data were correlated, expanded, and the resultant statistics adjusted to obtain the dependent variables (mean, standard deviation, and 1% flow). These statistics were then regressed to the physical characteristics of their respective basins (independent variables) by means of a mathematical process called Multiple Linear Regression. The regression equations chosen had minimal standard errors of estimates and maximum determination coefficients.

These equations are applicable to all unurbanized locations within the District to generate statistics for the determination of design flood peaks and volumes.

Gaged and Ungaged Locations

At gaged locations the statistics from the regression equations were compared with the site-specific statistics obtained from recorded data when such data were usable. The statistics from recorded data were obtained using the methods presented in the Water Resources Council Bulletin #17. If there were significant differences between the regional and site-specific results, then new statistics (weighted with respect to their respective standard errors) were calculated. These statistics, together with a Pearson TypeIII distribution a -0.6 regional skew coefficient or a weighted one, were used to determine the frequency regime.

At ungaged and rural locations, the results of the regression equations are considered to provide the best available information and, hence, are to be used for analysis and design.

Techniques Used for Developing Flood Hydrographs

Pattern runoff hydrographs are obtained from rainfall using Clark's Synthetic Unit Hydrograph method (as developed by the Corps of Engineers' HEC-1 computer program). This rainfall runoff transformation involves the determination of loss rates, synthetic unit hydrograph parameters, spatial distribution of rainfall over the waterhsed, and temporal distributions of rainfall rates within one storm. The first two items are usually obtained from projections of optimized results of recorded rainfall and its respective runoff data. The averaging of rainfall over the basin is usually done by the regular Theissen weighing operations. The temporal distribution is determined generally by using the Corps of Engineers' Standard Project Storm for Northern California.

If the peaks and volumes of the pattern hydrographs obtained are drastically different from the regional or site-specific results, then the reasons were investigated and, if justified, further adjustments were made. In this study, the rainfall-runoff-generated peaks and volumes are found to generally agree with regional or statistical values when average rainfall and area configurations were present. To avoid the uncertainties inherent in the rainfall-runoff transformation, the peaks and volumes generated from this transformation are balanced by regional values.

The balanced hydrographs obtained above are then combined with other hydrographs and routed downstream. The routing was done using linear methods (e.g., Muskingum by computing k when x = 0) if routing through channel system, or nonlinear methods (modified Pulse) if significant storage is available.

Rainfall distribution, time of concentration and the various routing operations are the items that define the shape of pattern hydrographs, with rainfall distribution being the foremost. The effect of the shape of these hydrographs, given constant peaks and volumes, may become significant when flow trapping is being considered as a flood protection measure. An in-depth analysis of the shape was not attempted as a part of this work.

Regulated Watersheds

In regulated watersheds, regulated by reservoirs, the design flows are assumed as inflows to the reservoirs. The intial storage levels of the reservoirs that coincide with the design flows of certain frequencies are computed from the stage-duration frequency (S.D.F.) analysis of daily recorded reservoir volumes. The Corps of Engineers' Coincidental Frequency Analysis (discussed in Leo Beard's "Statistical Methods") procedures were used to solve this compounded probability problem. At most reservoirs, the S.D.F. curves after 1968 were used because since that time the reservoirs have retained more water in storage longer to enhance recreation.

Urban Hydrology

The above procedures are applicable mainly to rural watersheds. For urban watersheds, the methodology changes and the regional regression equations do not apply.

Urban hydrology deals with the effects due to two major changes in the watershed, namely:

1. Increased imperviousness
2. Channelization

The increase in imperviousness is caused by rooftops, streets, parking lots, driveways, and other man-made ground surface cover. Channelization includes the flow of water in gutters, storm drains, improved channels, etc. An increase in the imperviousness tends to reduce the loss rates and increase the runoff volumes and peaks, while channelization may either increase or decrease the peaks. It depends on the magnitude of the flood and the storage-discharge capacity of the various channels or storm drain systems that carry that flood.

To determine the effect of increased imperviousness, the urban area of every watershed was divided into subareas tributary to the major storm drains. These subareas are then transformed into equivalent rectangular subareas where the length is equal to the length of an equivalent street. The ratio of imperviousness (which is a measure of land use) is then used to subdivide this equivalent subarea into two sub-subareas of pervious and impervious parts.

In order to calculate the separate inflow hydrographs from the two sub-subareas, the unit hydrograph procedure was employed using separate times of concentration for both the pervious and impervious parts. The routing coefficient is calculated from the length and width of these areas. The loss rates, which are the third set of parameters needed for the computation of the one percent flood hydrograph, are assumed to be 30 percent for the pervious part and five percent for the impervious part. Since most of the floodwater from urban areas is conveyed to flood control channels by the storm drain system, these hydrographs are routed through the storm drain system utilizing the modified Pulse routing method. This routing requires storage-discharge curves for the system of streets, gutters, manholes, and storm drains. Two actual urban areas, one of low-density residential and another of high-density residential and commercial, were used to develop equivalent generalized storage facilities and storage-discharge curves. The resultant curves were transformed to reflect unit areas, and then compared with each other. Since the two curves are very similar, an average smooth curve was chosen and used.

Because most of the urban area in Santa Clara Valley is in the flatland and similar storm drain design criteria were used throughout, it is assumed that the unit storage-discharge curve that was obtained from the above study is applicable to all the urban areas in the Valley.

The above method is detailed in a paper presented before the National Symposium on Urban Hydrology in 1976.

Upper 80 Percent Confidence Limits

As a District policy, the upper 80 percent confidence value for the 1% design flow rate is used as one input to define freeboard criteria. It is defined as the value at which there is a 90 percent chance that the design flow is equal to less than the given value. It relates to the standard errors associated with the statistical inferences of information about populations from very limited samples. These values are calculated using the Non-Central Student "t" Distribution as shown in the Water Resources Council Bulletin #17.

Previous Studies and Coordinated Results

As a part of the design flood flow determination procedure, all available hydrologic analyses done on any watershed were reviewed and considered. These previous analyses were sometimes the result of an in-house effort and other times the result of efforts of other agencies.

Because of the nature of random events, especially in the case of small samples, the results obtained from previous analyses were not necessarily congruent with the new values. Therefore, an in-depth search of the adequacy of the various results and methods involved was done.

As a result of this study, some of the adopted values were supported not only by District-developed methods and techniques but also by those of the Corps of Engineers and others.

Further Information

The attached information is only a summary of that available. Additional information, available upon request from the Hydrologic Library, is listed below:

1. Hydrographs, either plotted or numerical, at all locations.
2. Runoff from intermediate areas such as urban runoff.
3. Recorded streamflow data at most gaged locations.
4. Stochastically estimated data at most gaged stations.
5. A full spectrum of statistical analysis of the above data which include the investigation of outliers, the incorporation of historic flows, and the adjustment of statistics and probabilities.
6. Statistical analysis of rainfall data on all gaged locations.
7. Complete descriptions and documentation of computations done for each stream.
8. Other supporting documents of the methods like the 1976 Panel of Experts Report, the 1976 Regional Regression Study, Paper on Urban Hydrology, and Reports, by Zone, discussing the hydrology in detail.

GLOSSARY

Coincidental Initial Storage: The most likely initial storage found in a reservoir such that an outflowing event has almost the same frequency as an inflowing event.

Correlation: The association among sets of data having some mutual relation not necessarily cause and effect.

Correlation Coefficient: The measure of correlation between two sets of data.

Dependent Variable: In a regression equation, the variable (Y) that is expressed as a function of one or more independent variables (Xn).

Design Flood: The flood selected for design purposes assuming that the systems of creeks, channels, and rivers are all capable of conveying it without dealys, obstructions or overbank flooding. The water is contained within assumed design boundaries. It is measured in terms of design peak, design volume, and shape.

Determination Coefficient: The measure of percent of variance in the dependent variable explained by the independent variables. It is the correlation coefficient sqauared.

Flood: The movement of an amount of water that causes an overtopping of the natural banks. It is measured in terms of peaks, volumes, areas inundated, etc. The water is not necessarily contained within channel boundaries.

Flood Control Facility: Streams, creeks, rivers, reservoirs, etc., that help convey or detail floodwaters.

Flood Hydrograph: The flow patternof a flood in a time sequence.

Flood Traps; Flood control facilities designed to store the upper part of the inflowing hydrograph and reduce the peak flows with minimum storage availability.

Frequency Regime: The general understanding developed from frequency curves and confidence limits.

Independent Variable: In regression equations, the variables (Xn) on the right hand of the equation that represent the physical feature which could predict the dependent variable (y).

Maximum Annual Flow: The highest instantaneous peak flow in a water year. It is also called the Annual Flow.

Maximum Annual Flow Series: The continuous list of maximum annaul flows. It is also called Annual Flood Series.

One Percent Event (Flood, Design Flood, Peak Flow, Design Peak Flow, etc.): The event that has a one percent chance of being equaled or exceeded every year.

Optimization: The mathematical procedure used to find the most beneficial or least detrimental condition.

Pattern Hydrograph: The hydrograph of a flood generated from rainfall only. It reflects the rainfall-runoff transformation.

Peak Flow and Design Peak Flow: Peak flow is the instantaneous maximum flow rate in a flood. Design peak flow is the instantaneous peak flow in a design flood.

Population: The entire data from which samples are taken or collected.

Random Variable: The variable that is natural and unique. It is free from the influence of adjacent variables.

Regression: If the occurrence of Event Y depends on Event X, the functional relation between Y and X is called regression of Y on X.

Sample: A part of population. Hydrologic records are samples of longer population.

Standard Deviation: A measure of the dispersion or precision of a series of values such as precipitation, streamflows, etc.

Standard Error: An estimate of the standard deviation of the statistics of samples drawn from one population.

Statistics: The mathematical handling and manipulation of data. Statistics also refer to the statistical parameters like the mean, standard deviation and 1% value generated from an analysis.

Storm: The total amount of rain falling within a given duration.

Unit Hydrograph: The hydrograph with a volume of one inch (or one centimeter) of direct runoff from a storm of specified duration.

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Flint Creek, 4.9

Foothill Creek, 5.5, 5.9

Ford Ditch, 4.4

Fowler Creek, 4.9

Gavilan Creek, 5.2

Golf Creek, 3.6

Greystone Creek, 3.6

Guadalupe Creek, 3.6

Guadalupe River, 3.6, 3.7

Hale Creek, 1.6

Hayes Creek, 5.8

Jones Creek, 5.7, 5.11

Lions Creek, 5.6, 5.11

Little Arthur Creek, 5.1

Live Oak Creek, 5.6, 5.10

Llagas Creek, 5.4, 5.5, 5.6, 5.7, 5.8, 5.9, 5.10, 5.11

Los Coches Creek, 4.4

Los Gatos Creek, 3.1, 3.6

Los Trancos Creek, 1.1

Lower Penitencia Creek, 4.4, 4.5

Lower Silver Creek, 4.9, 4.10, 4.13

Machado Creek, 5.4, 5.8

Madrone Channel, 5.4, 5.9

Maple Creek, 5.5, 5.9

Matadero Creek, 1.3

Middle Fisher Creek, 4.13

Middle Fork Coyote Creek, 4.12

Miguelita Creek, 4.10

Miller Slough, 5.11

North Babb Creek, 4.9

North Branch Permanente Creek, 1.5

Norwood Creek, 4.9

Ortega Creek, 5.7, 5.11

Packwood Creek, 4.13

Palo Alto Flood Basin, 1.3

Panther Creek, 5.6, 5.10

Permanente Creek, 1.5, 1.6

Permanente Diversion, 1.6, 1.7

Piedmont Creek, 4.4

Princevale Drain, 5.7, 5.11

Prospect Creek, 2.1

Quimby Creek, 4.9

Randol Creek, 3.5

Regnart Creek, 2.1

Rodeo Creek, 2.1

Ronan Diversion, 5.6, 5.11

Ross Creek, 3.3, 3.6

Ruby Creek, 4.9

Rucker Creek, 5.6, 5.10

San Francisquito Creek, 1.1

San Martin Creek, 5.5, 5.9, 5.10

San Tomas Aquino Creek, 2.5

San Ysidro Creek, 5.7, 5.11

Santa Teresa Creek, 3.5

Saratoga Creek, 2.3, 2.5

Sierra Creek, 4.3

Skillet Creek, 5.5, 5.10

Smith Creek, 2.5

South Babb Creek, 4.9

South Branch Permanente Creek, 1.5

Stevens Creek, 1.5, 1.6

Sunny Hills Outfall, 4.5

Sweigert Creek, 4.3

Tar Creek, 5.2

Tennant Creek, 5.5, 5.9

Thompson Creek, 4.8, 4.9

Tick Creek, 5.2

Tularcitos Creek, 4.4

Upper Coyote Creek, 4.12

Upper Fisher Creek, 4.13

Upper Penitencia Creek, 4.1, 4.14

Upper Silver Creek, 4.13

Uvas Creek, 5.1, 5.2

West Branch Llagas Creek, 5.6, 5.10, 5.11

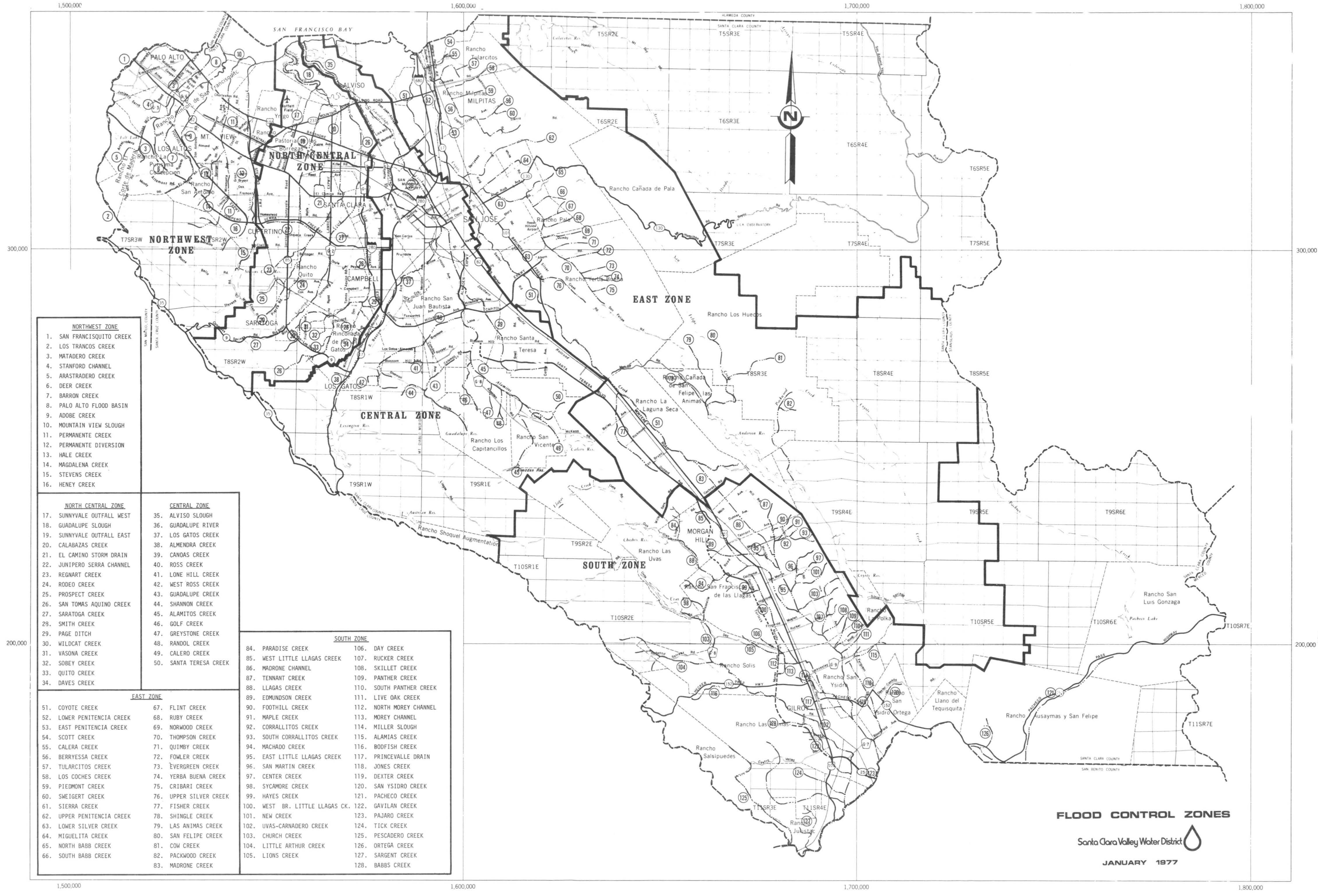
West Little Llagas Creek, 5.4, 5.8

Whitehurst Creek, 5.1

Wildcat Creek, 2.5

Wrigley Ditch, 4.4

Yerba Buena Creek, 4.8



NORTHWEST ZONE

1. SAN FRANCISQUITO CREEK
2. LOS TRANCOS CREEK
3. MATADERO CREEK
4. STANFORD CHANNEL
5. ARASTRADERO CREEK
6. DEER CREEK
7. BARRON CREEK
8. PALO ALTO FLOOD BASIN
9. ADOBE CREEK
10. MOUNTAIN VIEW SLOUGH
11. PERMANENTE CREEK
12. PERMANENTE DIVERSION
13. HALE CREEK
14. MAGDALENA CREEK
15. STEVENS CREEK
16. HENEY CREEK

NORTH CENTRAL ZONE

17. SUNNYVALE OUTFALL WEST
18. GUADALUPE SLOUGH
19. SUNNYVALE OUTFALL EAST
20. CALABAZAS CREEK
21. EL CAMINO STORM DRAIN
22. JUNIPERO SERRA CHANNEL
23. REGNART CREEK
24. RODEO CREEK
25. PROSPECT CREEK
26. SAN TOMAS AQUINO CREEK
27. SARATOGA CREEK
28. SMITH CREEK
29. PAGE DITCH
30. WILDCAT CREEK
31. VASONA CREEK
32. SOBEY CREEK
33. QUITO CREEK
34. DAVES CREEK

CENTRAL ZONE

35. ALVISO SLOUGH
36. GUADALUPE RIVER
37. LOS GATOS CREEK
38. ALMENDRA CREEK
39. CANOAS CREEK
40. ROSS CREEK
41. LONE HILL CREEK
42. WEST ROSS CREEK
43. GUADALUPE CREEK
44. SHANNON CREEK
45. ALAMITOS CREEK
46. GOLF CREEK
47. GREYSTONE CREEK
48. RANDOL CREEK
49. CALERO CREEK
50. SANTA TERESA CREEK

EAST ZONE

51. COYOTE CREEK
52. LOWER PENITENCIA CREEK
53. EAST PENITENCIA CREEK
54. SCOTT CREEK
55. CALERA CREEK
56. BERRYESSA CREEK
57. TULARCITOS CREEK
58. LOS COCHES CREEK
59. PIEDMONT CREEK
60. SWEIGERT CREEK
61. SIERRA CREEK
62. UPPER PENITENCIA CREEK
63. LOWER SILVER CREEK
64. MIGUELITA CREEK
65. NORTH BABB CREEK
66. SOUTH BABB CREEK
67. FLINT CREEK
68. RUBY CREEK
69. NORWOOD CREEK
70. THOMPSON CREEK
71. QUIMBY CREEK
72. FOWLER CREEK
73. EVERGREEN CREEK
74. YERBA BUENA CREEK
75. CRIBARI CREEK
76. UPPER SILVER CREEK
77. FISHER CREEK
78. SHINGLE CREEK
79. LAS ANIMAS CREEK
80. SAN FELIPE CREEK
81. COW CREEK
82. PACKWOOD CREEK
83. MADRONE CREEK

SOUTH ZONE

84. PARADISE CREEK
85. WEST LITTLE LLAGAS CREEK
86. MADRONE CHANNEL
87. TENNANT CREEK
88. LLAGAS CREEK
89. EDMUNDSON CREEK
90. FOOTHILL CREEK
91. MAPLE CREEK
92. CORRALLITOS CREEK
93. SOUTH CORRALLITOS CREEK
94. MACHADO CREEK
95. EAST LITTLE LLAGAS CREEK
96. SAN MARTIN CREEK
97. CENTER CREEK
98. SYCAMORE CREEK
99. HAYES CREEK
100. WEST BR. LITTLE LLAGAS CK.
101. NEW CREEK
102. UVAS-CARNADERO CREEK
103. CHURCH CREEK
104. LITTLE ARTHUR CREEK
105. LIONS CREEK
106. DAY CREEK
107. RUCKER CREEK
108. SKILLET CREEK
109. PANTHER CREEK
110. SOUTH PANTHER CREEK
111. LIVE OAK CREEK
112. NORTH MOREY CHANNEL
113. MOREY CHANNEL
114. MILLER SLOUGH
115. ALAMIAS CREEK
116. BODFISH CREEK
117. PRINCEVALLE DRAIN
118. JONES CREEK
119. DEXTER CREEK
120. SAN YSIDRO CREEK
121. PACHECO CREEK
122. GAVILAN CREEK
123. PAJARO CREEK
124. TICK CREEK
125. PESCADERO CREEK
126. ORTEGA CREEK
127. SARGENT CREEK
128. BABBS CREEK

FLOOD CONTROL ZONES

Santa Clara Valley Water District

JANUARY 1977

JAN 1979

Northwest

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: NORTHWEST

Major Watershed: SAN FRANCISQUITO CREEK AND TRIBUTARIES

Study Date: MARCH 1977

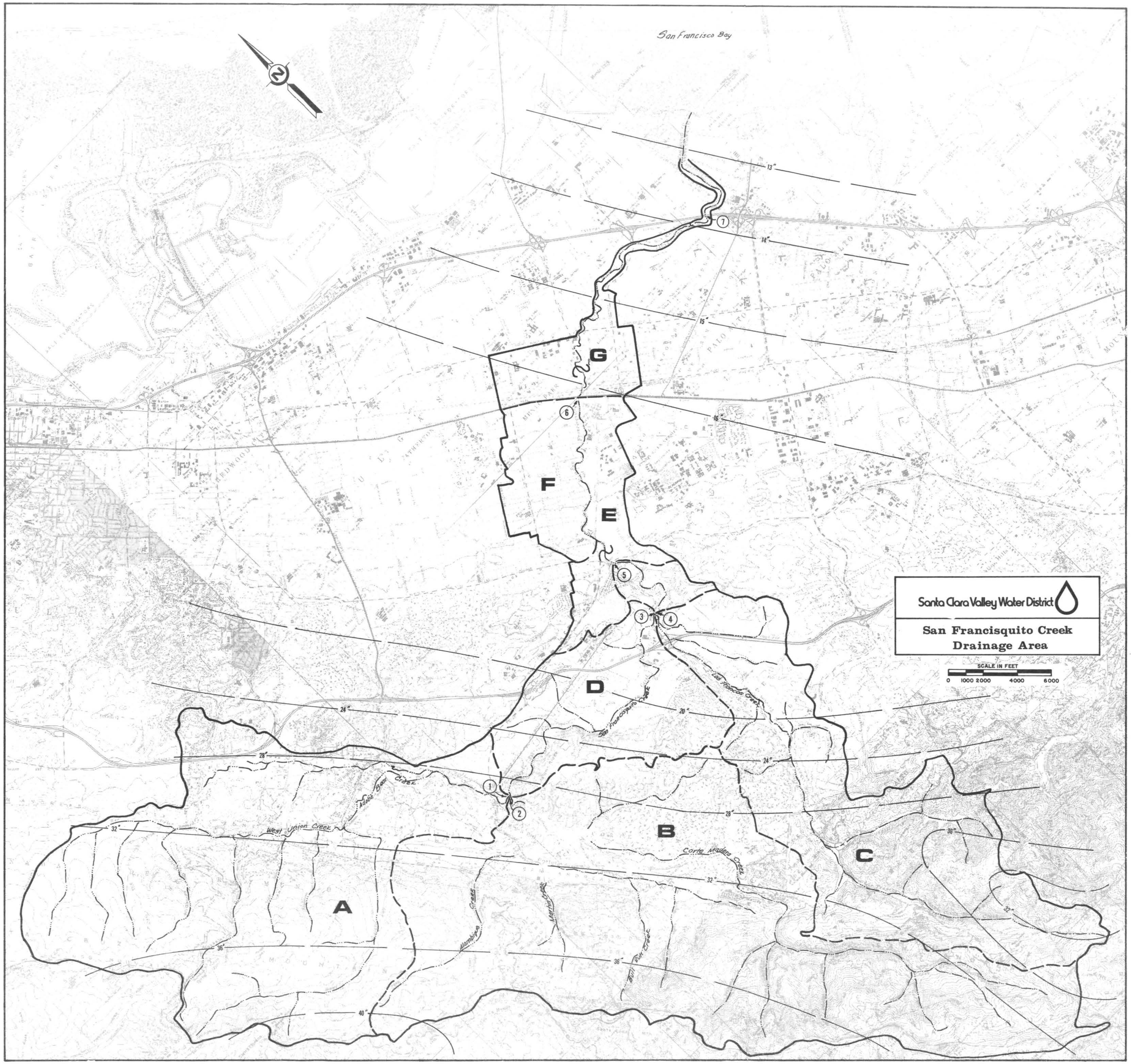
Assumptions:

1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with one-hour intervals.
3. Hydrographs balanced with the 1976 Regional Equations as modified based on 35 years of recorded data and 61 years of stocastically generated data at USGS Station Number 1645.
4. SCVWD Urban Hydrology Method used downstream of El Camino.
5. Lake Searsville Initial Storege = 800 AF.
6. All flows completely contained within the channel.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Bear Cr. U/S San Fransquito	1	12.24	3800	5000	2100	2600	1900	970
San Fran. U/S Lake Searsville	2	14.63	4100	5300	2300	2800	2100	1100
San Fran. D/S Lake Searsville	2	14.63	3600	4600	2000	2500	2000	1000
San Fran. D/S Bear Cr.	4	26.87	5600	7200	3200	3900	3300	1700
San Fran. U/S Los Trancos Cr.	4	30.13	6700	8800	3600	4500	3900	1900
Los Trancos Cr. U/S San Fran.	3	7.25	2300	3100	1200	1500	1000	510
San Fran. D/S Los Trancos Cr.	3	37.38	9000	12000	4800	6000	4900	2400
San Fran. @ USGS Station No. 1645	5	37.71	8600	11400	4400	5500	4900	2400
San Fran. @ El Camino	6	40.88	9100	12100	4800	6000	5300	2600
San Fran. D/S Middlefield	7	42.04	9300	12300	4900	6200	5400	2700

JAN 1979


N
San Francisco Bay
13"
14"
15"
16"
20"
24"
28"
32"
36"
40"
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7
San Francisquito Creek
Los Trancos Creek
Adobe Bear Creek
West Union Creek
Alambique Creek
Martin Creek
Bull Run Creek
Corte Madera Creek
Santa Clara Valley Water District
San Francisquito Creek Drainage Area
SCALE IN FEET
0 1000 2000 4000 6000

JAN 1979

Santa Clara Vall Water District
SUMMARY OF DESIGN FLOOD FLOWS

Pur .7 D mi.
370 1%

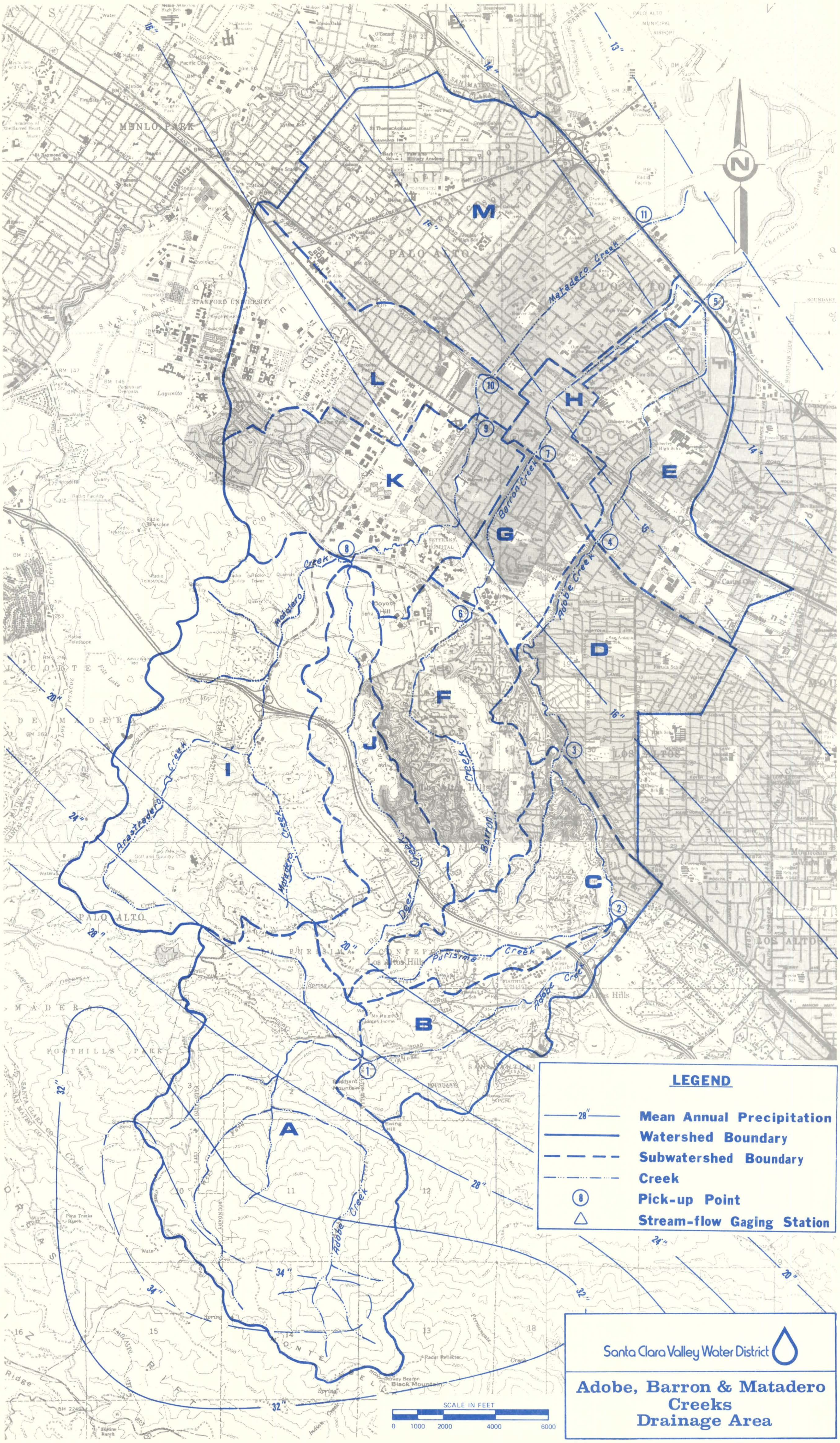
Zone: NORTHWEST
Major Watershed: ADOBE, MATADERO, AND BARRON CREEKS AND TRIBUTARIES
Study Date: MARCH 1977
Assumptions:

1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California
2. Hydrographs computed using HEC-1 with 30-minute intervals.
3. Hydrographs for rural areas balanced using 1976 Regional Equations.
4. SCVWD Urban Hydrology Method used downstream of Foothill Expressway.
5. All flows completely contained within the channel.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows 1% Peak (cfs)	1% 80% Conf (cfs)	10% Peak (cfs)	10% 80% Conf (cfs)	24-Hr. Volumes (Average Flow) 1% (cfs)	10% (cfs)
Adobe Cr. @ City Limits	1	4.25	1600	2000	820	1000	600	320
Adobe Cr. U/S Purissima Cr.	2	5.53	2000	2600	1100 170	1400	770	400
Adobe Cr D/S " "			2370		1270			
Adobe Cr. @ Foothill Expressway	3	7.10	2700	3500	1400	1800	980	490
Adobe Cr. @ El Camino	4	8.90	2900	3800	1600	2000	1200	630
Adobe Cr. U/S Barron Cr.	5	10.84	3100	4000	1800	2200	1400	780
Barron Cr. @ Foothill Expressway	6	1.76	760	1100	320	420	220	100
Barron Cr. @ El Camino	7	2.56	830	1200	410	540	310	160
Barron Cr. U/S Adobe Cr.	5	3.09	830	1200	460	600	360	200
Adobe Cr. D/S Barron Cr.	5	13.93	3900	5200	2300	2900	1700	980
Matadero Cr. U/S Deer Cr.	8	3.92	1500	2000	640	830	490	220
Deer Cr. U/S Matadero Cr.	8	1.60	740	1000	320	420	220	90
Matadero Cr. D/S Deer Cr.	8	5.52	2200	3000	960	1300	700	310
Matadero Cr. @ El Camino	9	7.55	2400	3300	1200	1500	910	460
Matadero Cr. @ Alma	10	9.42	2500	3500	1400	1800	1100	610
Matadero Cr. @ Bayshore	11	13.57	2900	4000	1800	2300	1600	930
Palo Alto Flood Basin Total In Flow	11	27.50	6800	9300	4100	5200	3300	1900

JAN 1979



Adobe, Barron & Matadero Creeks Drainage Area

JAN 1979

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: NORTHWEST

Major Watershed: STEVENS CREEK, PERMANENTE CREEK AND TRIBUTARIES

Study Date: FEBRUARY 1978

Assumptions:

1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 30-minute intervals.
3. Rural hydrographs balanced using 1976 Regional Equations.
4. Santa Clara Valley Water District Urban Hydrology Method used downstream of Highway 280.
5. Stevens Creek Reservoir initial storage = 3595 acre feet (full).
6. All flows completely contained within the channel.
7. Permanente Diversion is assumed to contribute a maximum of 1700 cfs to Stevens Cr.

TABLE OF DESIGN FLOOD FLOWS

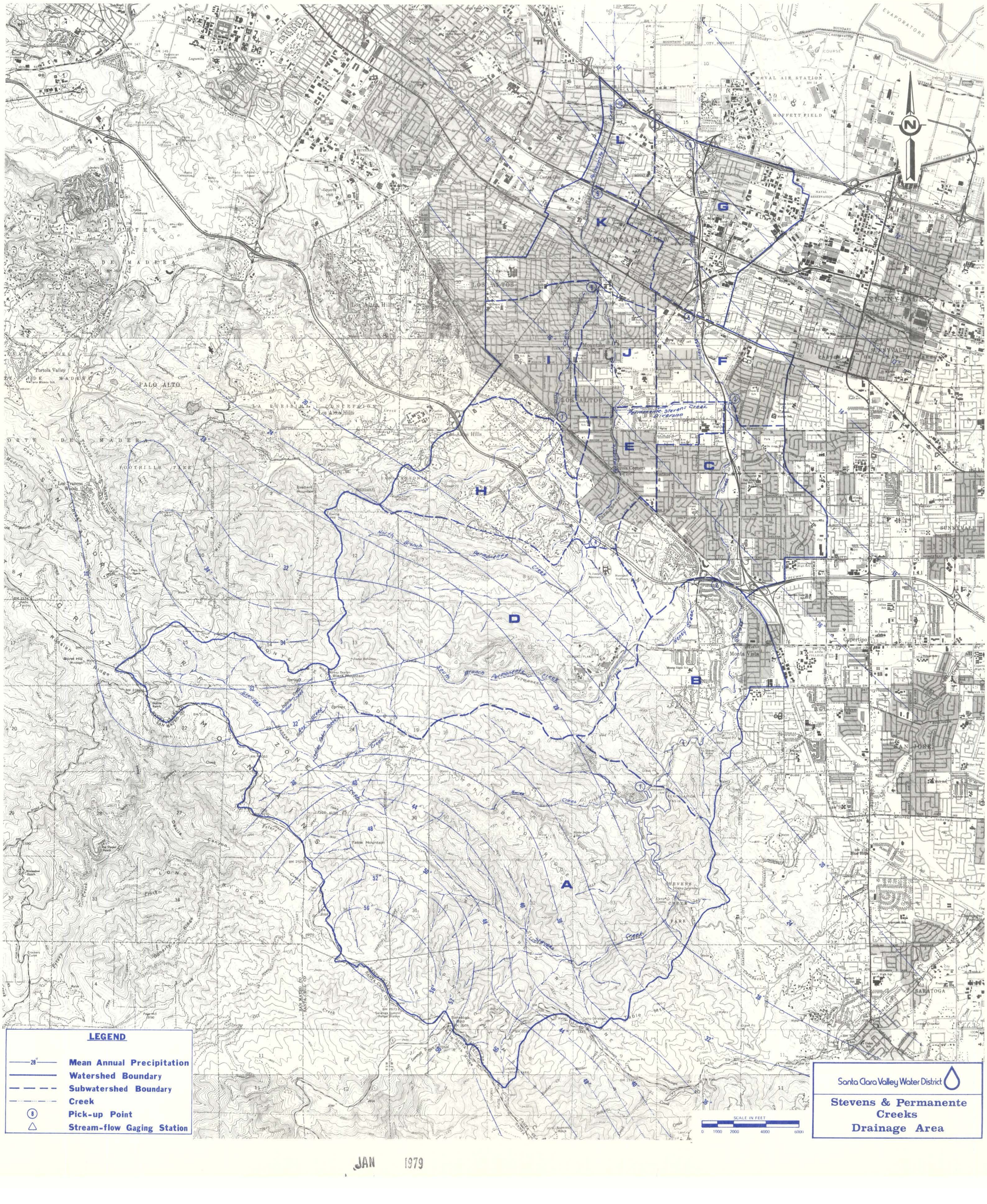
Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Stevens Creek Upstream Reservoir	1	17.30	6000	7800	2900	3600	2700	1400
Stevens Creek Downstream Reservoir	1	17.30	5500	7100	2800	3500	2500	1300
Stevens Creek at Highway 280	2	20.20	6000	7800	3100	3800	2800	1400
Stevens Cr. U/S Permanente Diversion	3	24.63	6400	8400	3400	4300	3200	1600
N. Branch Permanente U/S Hwy 280	4	3.62	1400	1900	700	880	510	250
S. Branch Permanente U/S Hwy 280	4	4.20	1700	2200	830	1000	620	300
Permanente Creek at Highway 280	4	7.82	2600	3400	1400	1800	1000	680
Permanente Creek at Diversion	3	8.96	2800	3700	1500	1900	1200	750
Permanente Diversion to Stevens Cr.	3	8.96	1700	2200	1100	1400	690	480
Stevens Creek Downstream Diversion	3	33.59	7900	10300	4500	5600	3900	2100

JAN 1979

STEVEN :REEK,PERMANENTE CREEK AND TRIBUTARIES (:inued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Stevens Creek at El Camino	5	35.10	8000	10500	4600	5800	4000	2200
Stevens Creek at Bayshore	6	38.04	8300	10900	4900	6100	4300	2400
Hale Cr. at Fremont S.C. #38	7	2.62	1100	1500	460	610	340	150
Hale Creek Upstream Permanente	8	4.82	1100	1600	640	840	580	300
Permanente Creek D/S Diversion	3	0.00	1100	1400	480	620	470	240
Permanente Creek Upstream Hale Cr.	8	1.15	1200	1600	600	770	590	310
Permanente Creek Downstream Hale Cr.	8	5.97	2400	3200	1200	1600	1200	610
Permanente Creek at SPRR	9	7.35	2300	3100	1300	1700	1300	690
Permanente Creek at Bayshore	10	8.11	2400	3200	1400	1800	1400	740



LEGEND

—28— Mean Annual Precipitation

——— Watershed Boundary

– – – Subwatershed Boundary

–·–·– Creek

⑧ Pick-up Point

△ Stream-flow Gaging Station

Santa Clara Valley Water District

Stevens & Permanente Creeks Drainage Area

JAN 1979

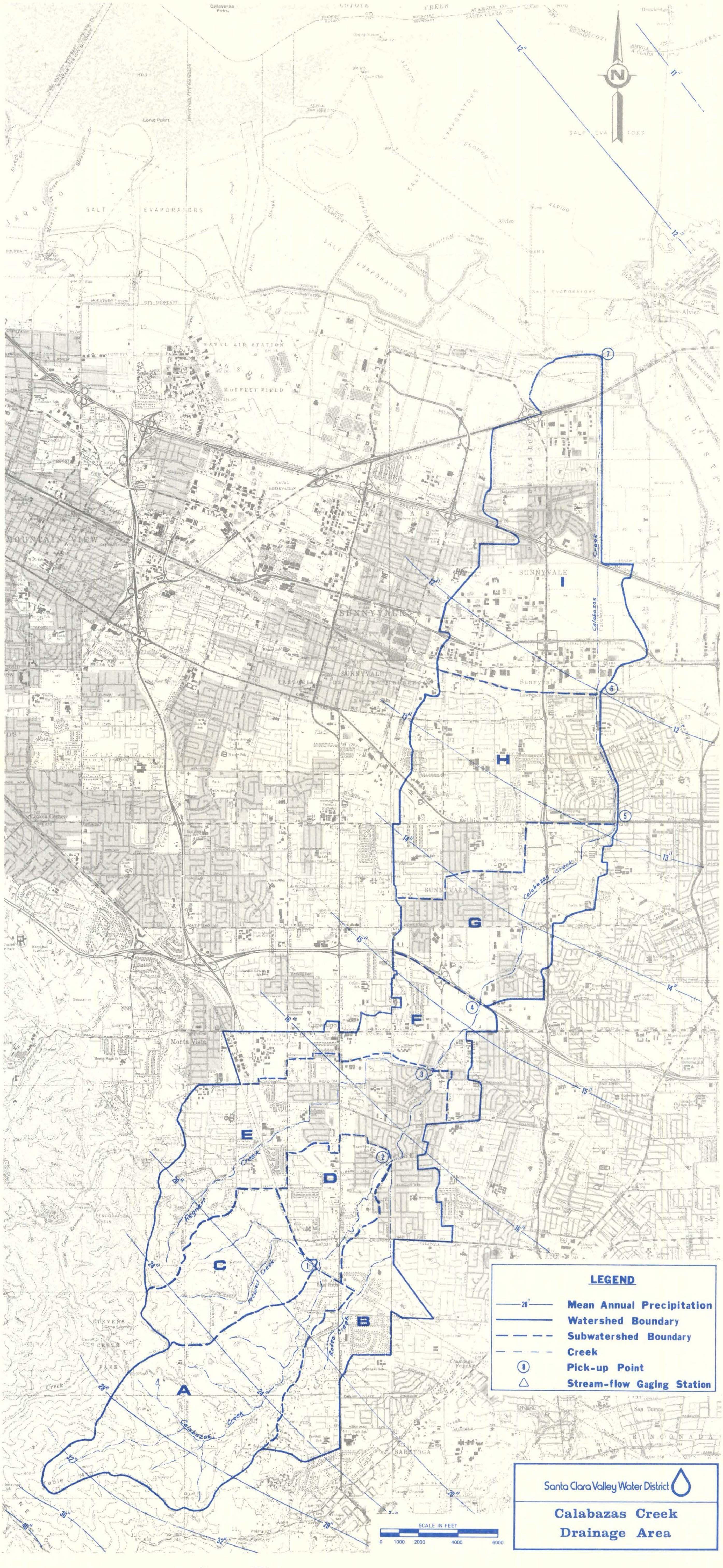
North Central

Santa Clara Vall Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: NORTH CENTRAL
Major **Watershed:** CALABAZAS CREEK AND TRIBUTARIES
Study Date: AUGUST 1977
Assumptions:
1. 24 hr. storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 15-minute intervals.
3. Rural Hydrographs balanced with 1976 Regional Equations.
4. Santa Clara Valley Water District Urban Hydrology Method used downstream of Rodeo Creek.
5. All flows completely contained within the channel (n = .03).
6. **All flows computed by G. S. Nolte and Associates using SCVWD methodology.**

TABLE OF DESIGN FLOOD FLOWS

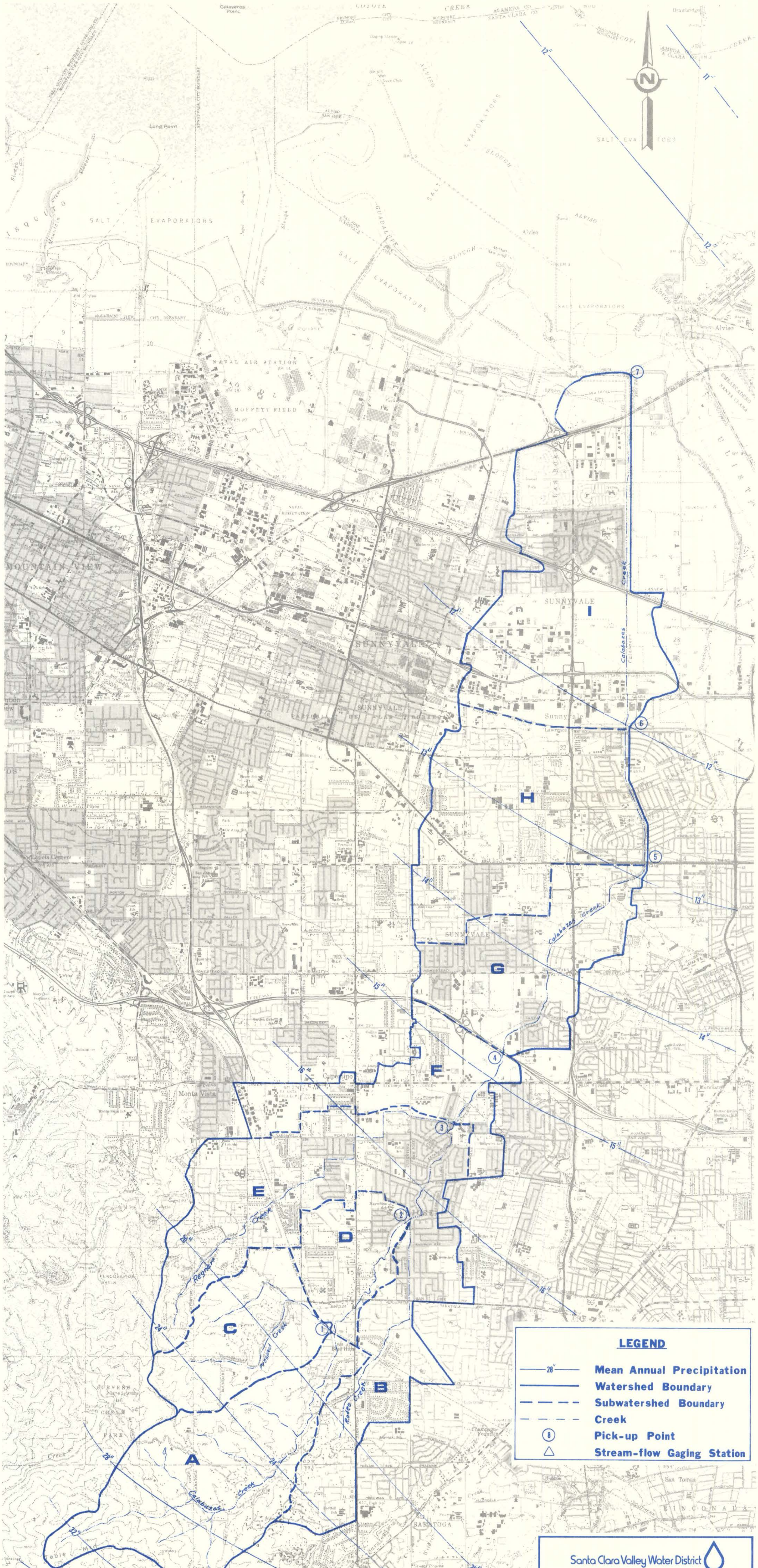
Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flow)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Prospect Cr. U/S Calabazas Creek	1	1.44	630	840	300	380	190	90
Calabazas Cr. U/S Prospect Creek	1	2.85	1200	1500	580	730	410	20
Calabazas Cr. D/S Prospect Creek	1	4.29	1800	2400	880	1100	600	300
Calabazas Cr. U/S Rodeo Creek	2	5.25	1900	2500	960	1200	730	380
Rodeo Cr. U/S Calabazas Creek	2	1.43	320	420	260	320	210	120
Calabazas Cr. D/S Rodeo Creek	2	6.68	2100	2800	1200	1500	940	490
Regnart Cr. U/S Calabazas Creek	3	3.41	690	920	520	650	470	280
Calabazas Cr. D/S Regnart	3	10.09	2600	3500	1600	2000	1400	770
Calabazas Cr. @ Hwy. 280	4	11.59	3000	4000	1900	2400	1600	890
Calabazas Cr. @ El Camino Real	5	13.70	3100	4100	2000	2500	1800	1000
El Camino Storm Drain U/S Calabazas	6	3.23	660	880	520	660	380	220
Calabazas Cr. @ SPRR	6	16.93	3800	5000	2500	3100	2200	1300
Calabazas Cr. D/S Hwy 237 @ Pump Station	7	20.67	4300	5700	2800	3500	2600	1500



Santa Clara Valley Water District

Calabazas Creek Drainage Area

JAN 1979


LEGEND
28" Mean Annual Precipitation
Watershed Boundary
Subwatershed Boundary
Creek
8 Pick-up Point
△ Stream-flow Gaging Station
Santa Clara Valley Water District
Calabazas Creek
Drainage Area
SCALE IN FEET
0 1000 2000 4000 6000
N
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Calabazas Creek
Regnart Creek
Prospect Creek
Rodeo Creek
12"
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24"
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32"
36"
40"
11"
SUNNYVALE
MOUNTAIN VIEW
NAVAL AIR STATION
MOFFETT FIELD
SALT EVAPORATORS
Alviso
Cupertino
Monta Vista
SARATOGA
San Tomas
Long Point
Calaveras Point

JAN 1979

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

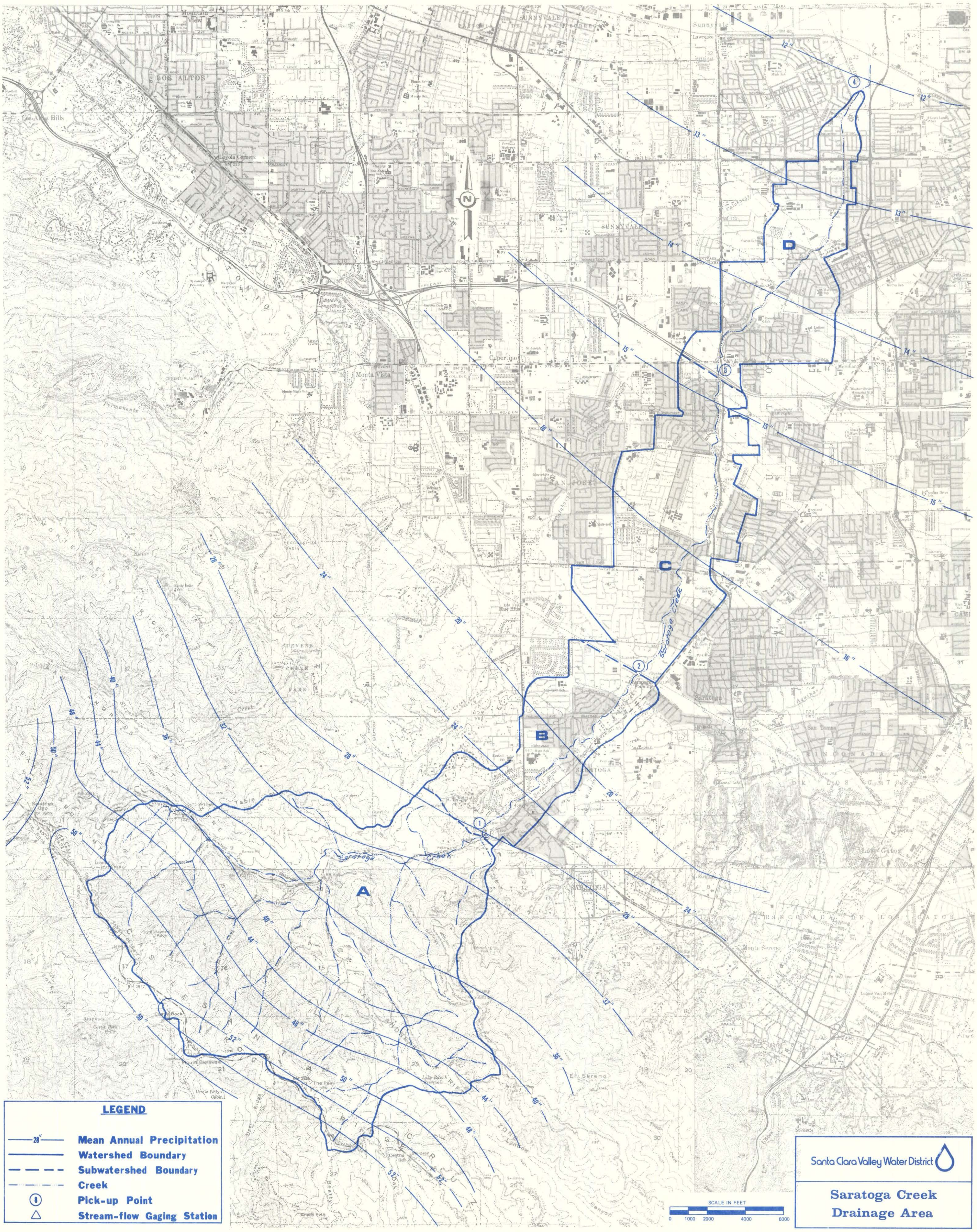
Zone: NORTH CENTRAL
Major Watershed: SARATOGA CREEK AND TRIBUTARIES
Study Date: MARCH 1977
Assumptions:

1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 30-minute intervals.
3. Rural hydrographs balanced using 1976 Regional Equations.
4. Santa Clara Valley Water District Urban Hydrology Method used downstream of USGS gage.
5. All flows completely contained within the channel.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Saratoga Creek at USGS Gage	1	9.20	3500	4400	1900	2300	1600	890
Saratoga Creek at SPRR	2	11.16	3600	4500	2200	2600	1800	1100
Saratoga Creek at Highway 280	3	14.32	3900	4900	2500	3000	2200	1300
Saratoga Creek u/s San Tomas Creek	4	16.56	4100	5100	2700	3300	2400	1500

JAN 1979



LEGEND

- ——28"—— **Mean Annual Precipitation**
- ——— **Watershed Boundary**
- — — — **Subwatershed Boundary**
- —·—·— **Creek**
- ⑧ **Pick-up Point**
- △ **Stream-flow Gaging Station**

Santa Clara Valley Water District

Saratoga Creek
Drainage Area

JAN 1979

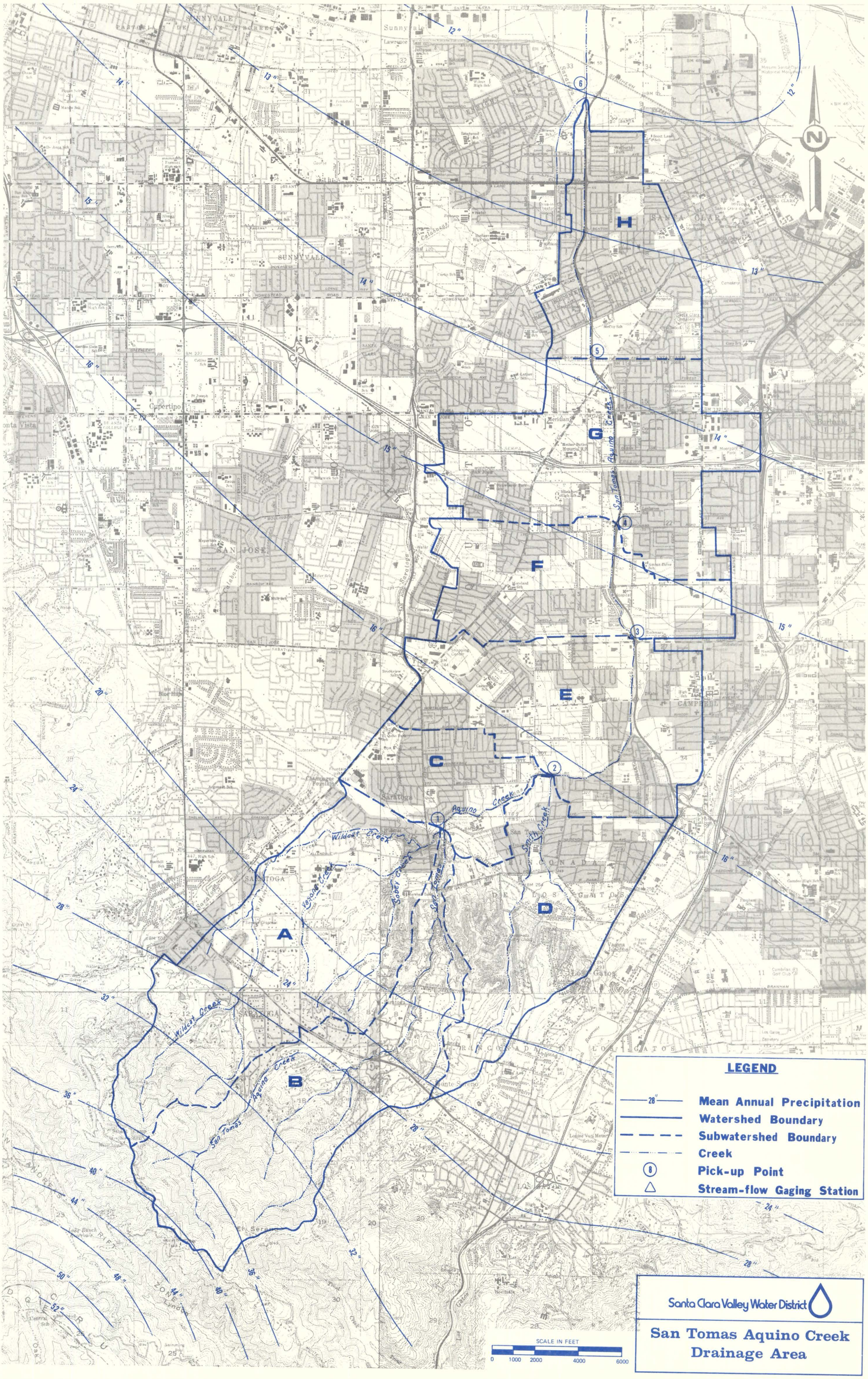
Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: NORTH CENTRAL
Major Watershed: SAN TOMAS AQUINO CREEK AND TRIBUTARIES
Study Date: MARCH 1977
Assumptions:
1. 24 hr. storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 30-minute intervals.
3. Rural hydrographs balanced using 1976 Regional Equations.
4. Santa Clara Valley Water District Urban Hydrology Method used downstream of Wildcat Creek confluence.
5. All flows completely contained within the channel.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flow)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
San Tomas Cr. U/S Wildcat Creek	1	3.64	1600	2000	790	980	580	290
Wildcat Cr. U/S San Tomas Creek	1	4.12	1600	2100	760	960	570	270
San Tomas Cr. D/S Wildcat Creek	1	7.76	3100	4100	1500	1900	1100	560
San Tomas Cr. U/S Smith Creek	2	8.99	3100	4100	1700	2100	1300	660
Smith Cr. U/S San Tomas Creek	2	2.65	1000	1400	440	580	320	140
San Tomas Cr. D/S Smith Creek	2	11.64	4100	5500	2200	2700	1600	800
San Tomas Cr. @ Hamilton Avenue	3	14.72	4400	5800	2400	3100	1900	1000
San Tomas Cr. @ Williams Road	4	16.91	4600	6200	2700	3400	2200	1200
San Tomas Cr. @ Pruneridge Avenue	5	20.40	4900	6600	3000	3900	2500	1500
San Tomas Cr. U/S Saratoga Creek	6	22.50	5100	6800	3200	4100	2700	1600
Saratoga Cr. U/S San Tomas Creek	6	16.56	4100	5100	2700	3300	2400	1500
San Tomas Cr. D/S Saratoga Creek	6	29.06	9100	11800	5900	7400	5100	3100

JAN 1979



Santa Clara Valley Water District

San Tomas Aquino Creek Drainage Area

JAN 1979

Central

Santa Clara Valley Water District
SUMMARY OF DES FLOOD FLOWS

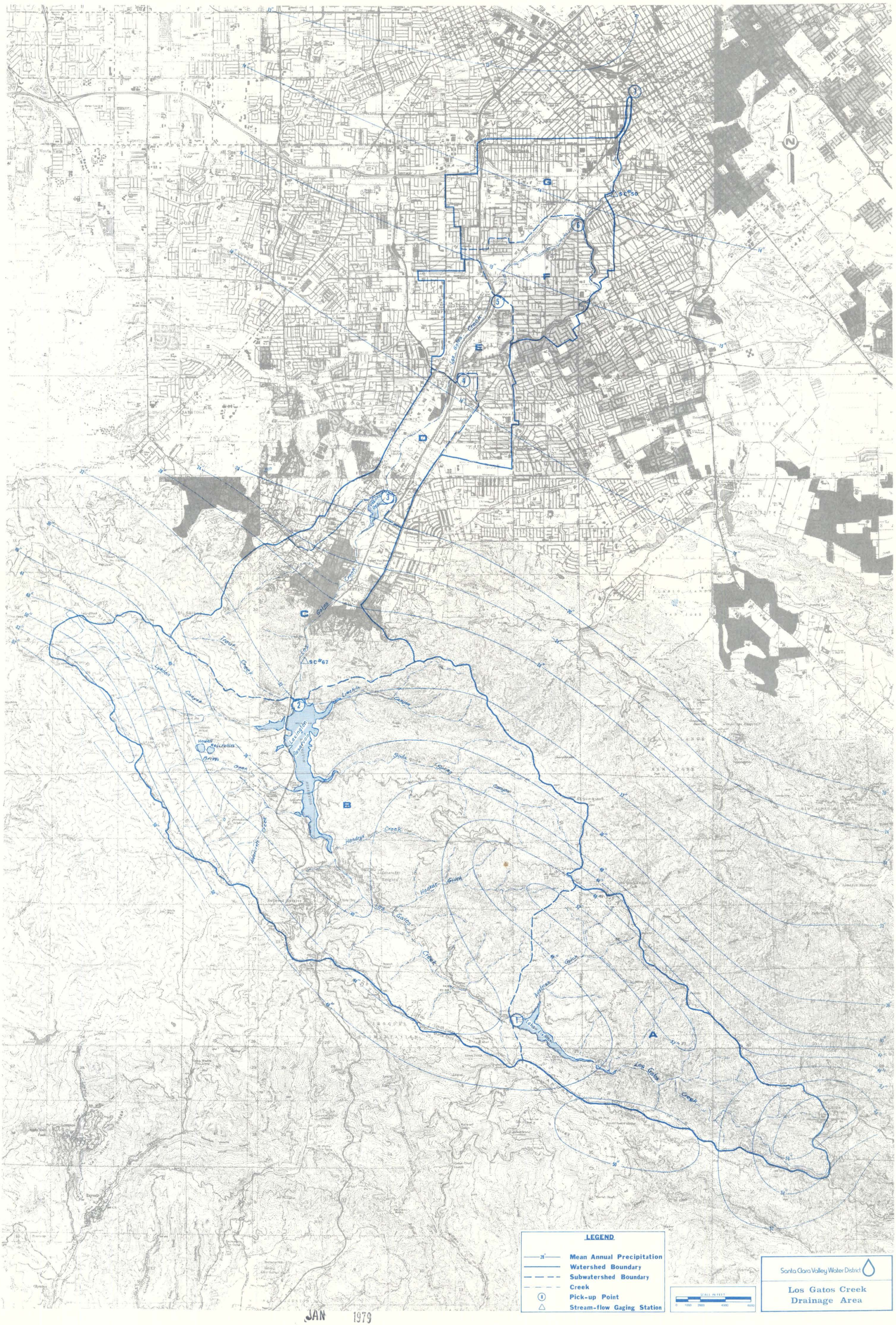
Zone: CENTRAL
Major Watershed: LOS GATOS CREEK AND TRIBUTARIES
Study Date: MARCH 1977
Assumptions:

1. 24 hr. storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 1-hour intervals.
3. Rural hydrographs balanced with the 1976 Regional Equations.
4. Santa Clara Valley Water District Urban Hydrology Method used downstream of Camden Ave.
5. Vasona Reservoir and Lake Elsman assumed full. Lexington coincidental initial storage = 14,000 AF for 1% flows and 15,000 AF for 10% flows.
6. All flows completely contained within the channel.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows 1% Peak (cfs)	Peak Design Flows 1% 80% Conf (cfs)	Peak Design Flows 10% Peak (cfs)	Peak Design Flows 10% 80% Conf (cfs)	24-Hr. Volumes (Average Flow) 1% (cfs)	24-Hr. Volumes (Average Flow) 10% (cfs)
Inflow to Lake Elsman	1	9.91	4100	5100	2400	2800	2000	1200
Outflow from Lake Elsman	1	9.91	3400	4300	2100	2500	1900	1100
Inflow to Lexington Reservoir	2	37.52	11200	14100	6300	7500	6000	3300
Outflow from Lexington Reservoir	2	37.52	6600	8400	1600	1900	3300	950
Inflow to Vasona Reservoir	3	43.98	7000	8800	1600	1900	3600	1000
Outflow from Vasona Reservoir	3	43.98	7000	8800	1600	1900	3600	1000
Los Gatos Cr. @ Camden Avenue	4	46.15	7000	8800	1500	1900	3800	1100
Los Gatos Cr. @ Hwy. 17	5	48.79	7300	9200	1600	1900	4000	1200
Los Gatos Cr. @ Meridian Ave.	6	51.24	7600	9500	1800	2200	4200	1300
Los Gatos Cr. U/S Guadalupe River	7	54.83	8000	10000	2200	2600	4500	1400

JAN 1979



Santa Clara Valley Water District

Los Gatos Creek Drainage Area

JAN 1979

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

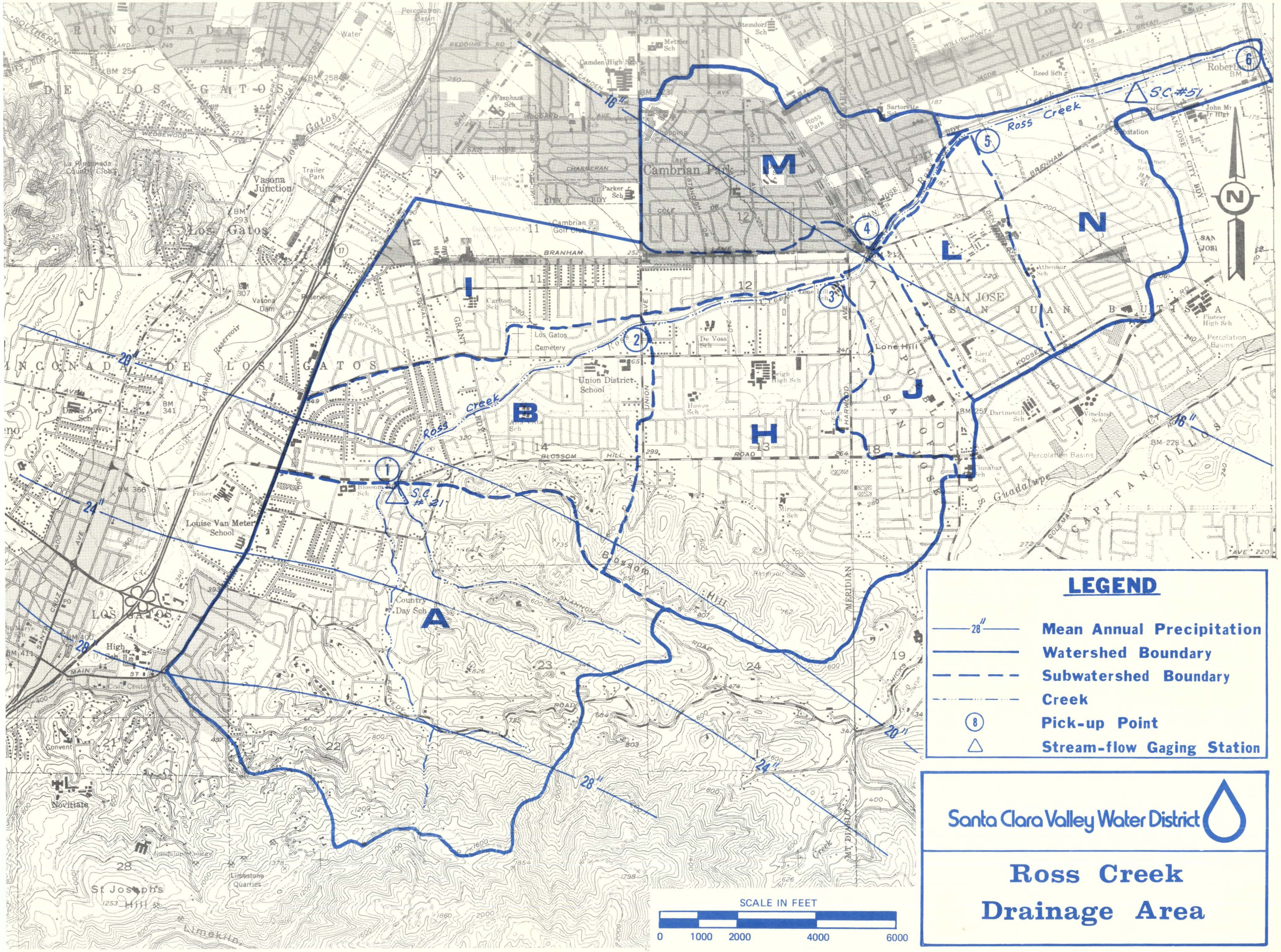
Zone: CENTRAL
Major Watershed: ROSS CREEK
Study Date: MARCH 1976
Assumptions:

1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 15-minute intervals.
3. Hydrographs for rural areas balanced using 1976 Regional Equations.
4. SCVWD Urban Hydrology Method used downstream of Blossom Hill Road.
5. All flows completely contained within the channel.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Ross Creek at Blossom Hill Road (Gage 21)	1	2.86	1100	1500	530	680	480	200
Ross Creek at Union Avenue	2	4.01	1300	1700	650	830	600	300
Ross Creek at Harwood Avenue	3	5.97	1500	2000	860	1100	810	450
Ross Creek at Camden Avenue	4	7.45	1700	2200	1000	1300	980	570
Ross Creek at Meridian Avenue	5	9.01	1800	2500	1200	1500	1200	690
Ross Creek Upstream Guadalupe River	6	9.96	1900	2600	1300	1600	1300	760

JAN 1979



Santa Clara Valley Water District

Ross Creek Drainage Area

JAN 1979

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: CENTRAL
Major Watershed: GUADALUPE RIVER AND TRIBUTARIES
Study Date: OCTOBER 1977
Assumptions:
1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 30-minute intervals.
3. Hydrographs for rural areas balanced using 1976 Regional Equations.
4. SCVWD Urban Hydrology Method used downstream of Alamitos Creek confluence.
5. For the 1% flows, coincidental initial reservoir storages were 9300 acre-feet for Calero, 800 acre-feet for Almaden and 2900 acre-feet for Guadalupe. For the 10% flows, coincidental initial reservoir storages were 9300 acre-feet for Calero, 900 acre-feet for Almaden and 3150 acre-feet for Guadalupe.
6. All flows completely contained within the channels.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volume (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Inflow to Calero Reservoir	1	6.99	2600	3500	1200	1500	970	450
Outflow from Calero Reservoir	1	6.99	630	860	180	230	420	180
Calero Cr. Upstream Santa Teresa Cr.	2	9.64	1300	1800	660	830	660	330
Santa Teresa Cr. Upstream Calero Cr.	2	1.99	860	1200	360	470	250	110
Calero Cr. Downstream Santa Teresa Cr	2	11.63	2200	3000	1000	1300	900	440
Calero Creek Upstream Alamitos Creek	4	12.39	2300	3100	1200	1500	990	480
Inflow to Almaden Reservoir	3	12.00	4400	5700	2300	2800	1800	970
Outflow from Almaden Reservoir	3	12.00	3500	4500	1300	1500	1300	500
Alamitos Creek Upstream Calero Creek	4	16.24	4700	6100	1400	1800	1800	720
Alamitos Creek Downstream Calero Cr.	4	28.63	6800	9100	2200	2700	2800	1200
Alamitos Creek Upstream Randol Creek	5	29.37	6800	9100	2300	2900	2800	1200
Randol Creek Upstream Alamitos Creek	5	2.28	930	1300	410	540	290	130
Alamitos Creek Downstream Randol Cr.	5	31.65	7400	10000	2700	3400	3100	1400
Alamitos Creek Upstream Greystone Cr.	6	32.42	7400	10100	2700	3500	3200	1400

JAN 1979

(continued)

GUADALUPE RIVER AND TRIBUTARIES (continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows 1% Peak (cfs)	1% 80% Conf (cfs)	10% Peak (cfs)	10% 80% Conf (cfs)	24-Hr. Volumes (Average Flows) 1% (cfs)	10% (cfs)
Greystone Creek Upstream Alamitos Cr.	6	1.38	640	880	280	360	180	80
Alamitos Creek Downstream Greystone Cr	6	33.80	7800	10500	2900	3700	3400	1500
Golf Creek Upstream Alamitos Creek	7	3.08	1200	1600	500	660	380	160
Alamitos Creek Downstream Golf Creek	7	37.41	8700	11800	3500	4500	3800	1700
Alamitos Creek Upstream Guadalupe Cr.	10	37.96	8900	12100	3600	4700	3900	1700
Inflow to Guadalupe Reservoir	8	5.96	2300	3100	1200	1500	840	440
Outflow from Guadalupe Reservoir	8	5.96	920	1200	230	290	400	230
Guadalupe Creek Downstream Shannon Cr.	9	12.71	2800	3600	1500	1900	1200	670
Guadalupe Creek Upstream Alamitos Cr.	10	15.24	3000	3900	1700	2100	1500	800
Guadalupe River Downstream Alamitos Cr	10	53.20	11800	16000	5300	6800	5400	2500
Guadalupe River Upstream Ross Creek	2	55.29	12000	15300	5400	7000	5600	2600
Ross Creek Upstream Guadalupe River	2	9.96	1900	2600	1300	1600	1300	760
Guadalupe River Downstream Ross Creek	2	65.25	13900	17500	6700	8600	6900	3400
Guadalupe River Upstream Canoas Creek	3	69.98	14300	18000	7200	9200	7400	3700
Canoas Creek at Cottle Road	4	4.56	510	610	480	560	490	330
Canoas Creek at Santa Teresa Blvd.	5	7.41	830	990	780	900	790	510
Canoas Creek at Blossom Hill Road	6	12.51	1400	1700	1300	1500	1300	900
Canoas Creek at Capitol Expressway	7	17.55	2000	2300	1900	2100	1900	1300
Canoas Creek Upstream Guadalupe River	3	18.62	2400	2800	2000	2300	2000	1300
Guadalupe River Downstream Canoas Cr.	3	88.60	16500	21900	9200	11600	9400	5100
Guadalupe River at West Alma Avenue	8	92.06	16500	22000	9500	12000	9700	5300
Guadalupe River Upstream Los Gatos Cr.	9	95.30	16500	21900	9800	12300	10100	5600
Los Gatos Creek Upstream Guadalupe Riv	9	54.83	8000	10000	2200	2600	4400	1400
Guadalupe River Downstream Los Gatos Cr	9	150.13	19800	26100	11900	15000	14100	6800

JAN 1979

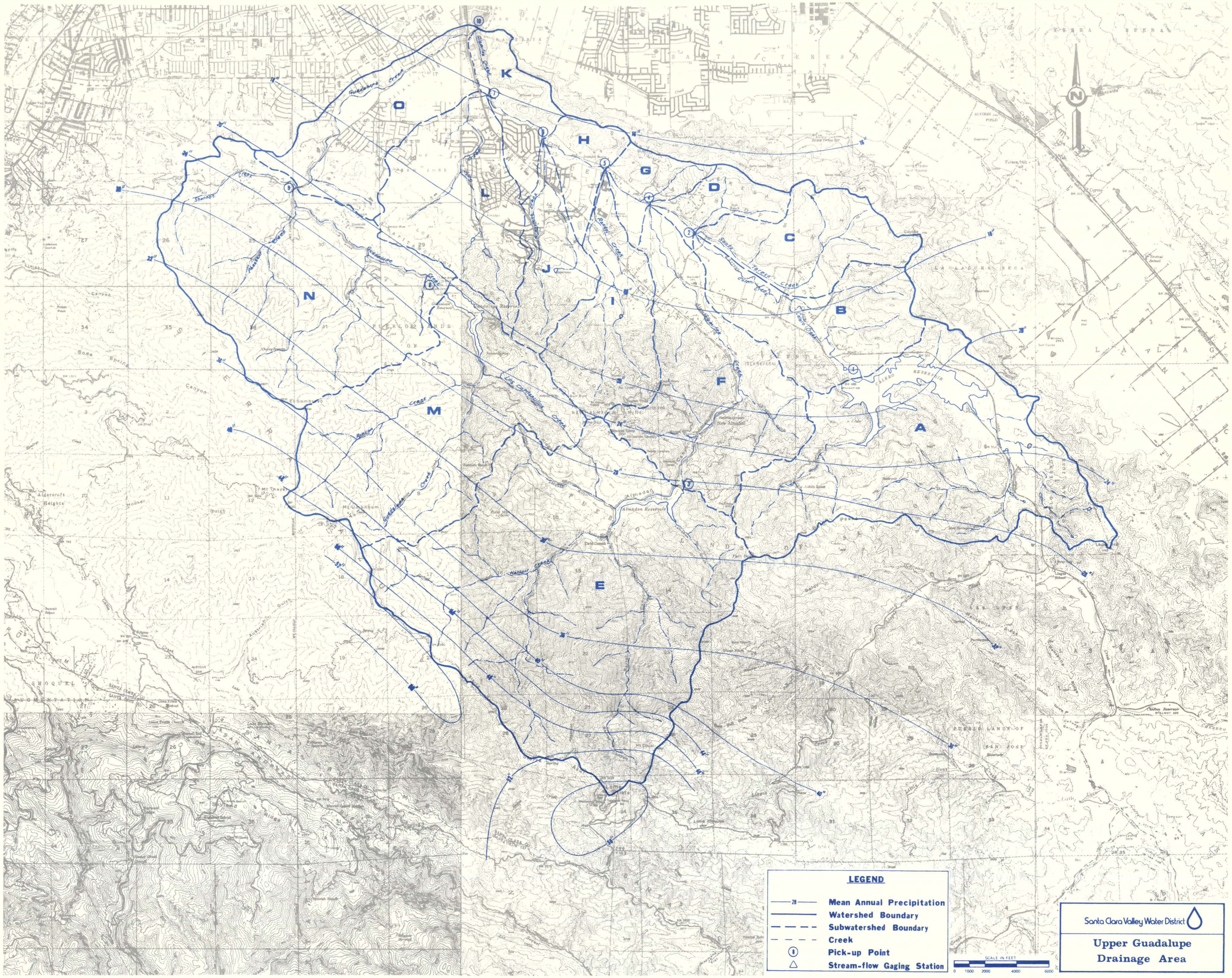
Comp. by Sach 23 June 8[illegible]

(continued)

GUADALUPE RIVER AND TRIBUTARIES (continued)

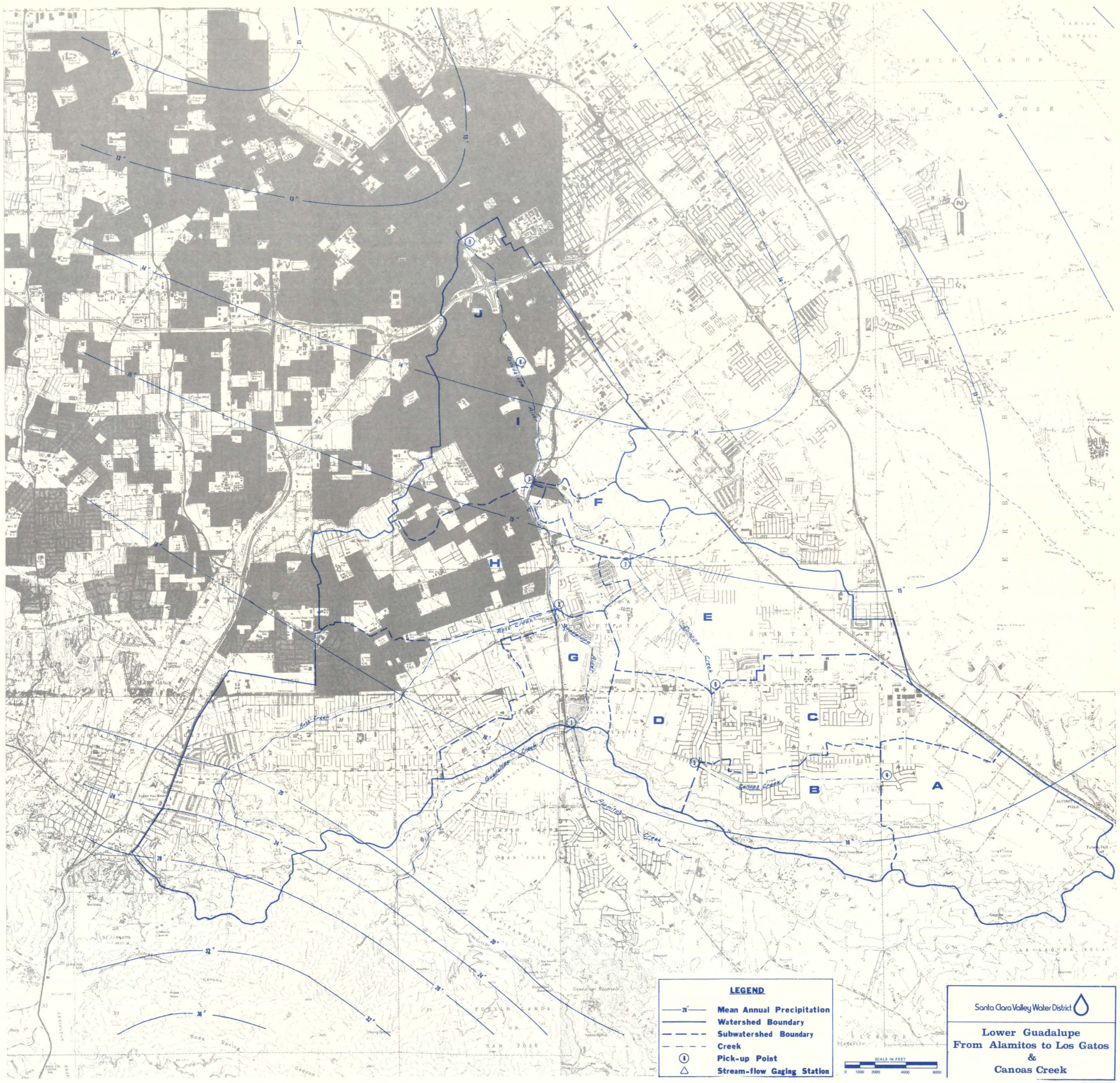
TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Guadalupe River at Highway 17	10	154.12	20200	26700	12200	15400	14500	7100
Guadalupe River at Highway 101	11	160.25	20800	27400	12800	16100	15100	7500
Guadalupe River at Highway 237	12	170.64	21700	28600	13700	17200	16100	8100



Upper Guadalupe Drainage Area

JAN 1979



Santa Clara Valley Water District

Lower Guadalupe
From Alamitos to Los Gatos
&
Canoas Creek

JAN 1979

East

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

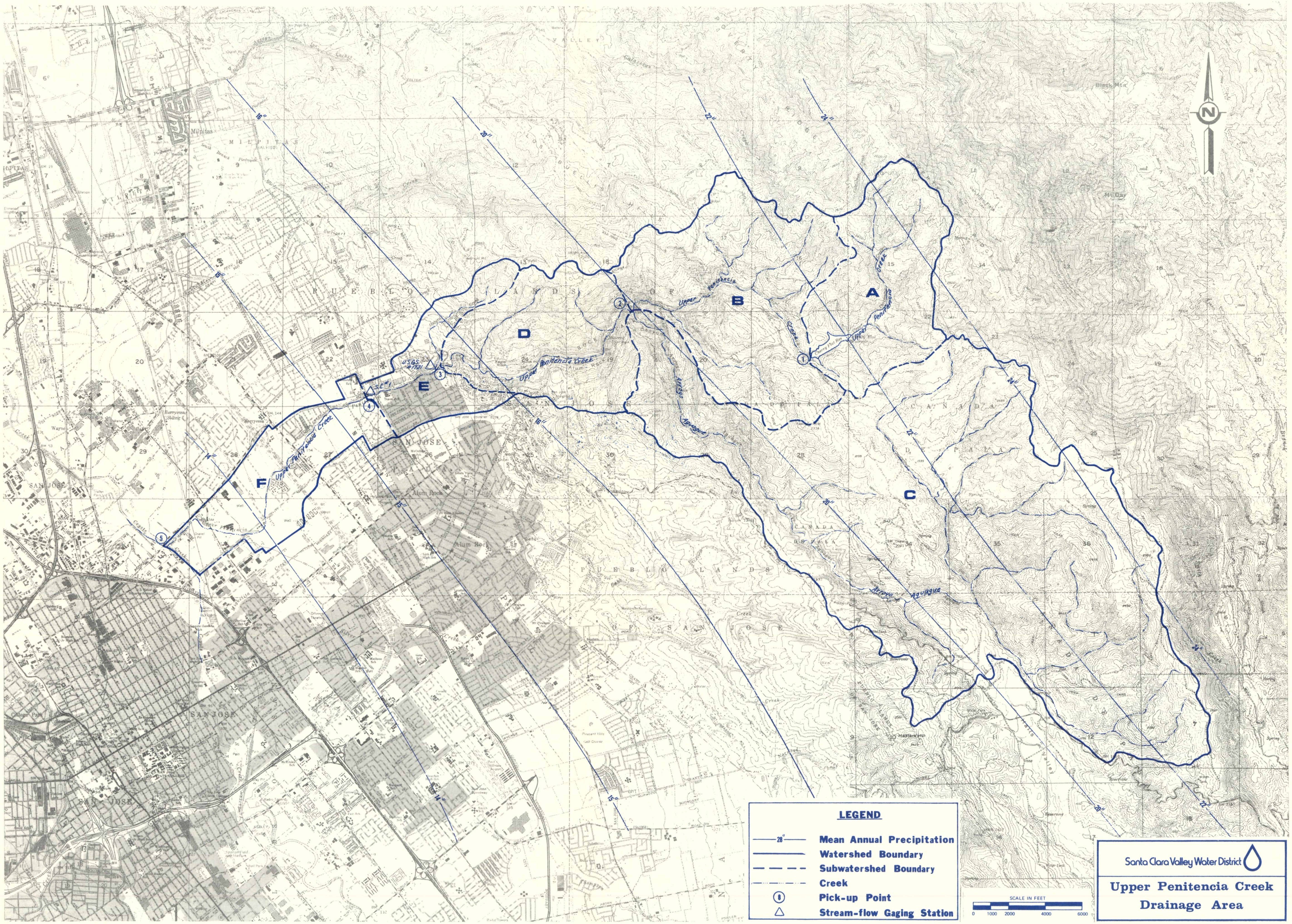
Zone: EAST
Major Watershed: UPPER PENITENCIA CREEK
Study Date: SEPTEMBER 1977
Assumptions:

1. 24-hour storm distributed using Army Corps of Engineers 1955 Storm Distribution for Hollister, Freedom and Stayton Mine (PL 18).
2. Hydrographs computed using HEC-1 with 15-minute intervals.
3. Hydrographs at USGS #1721 balanced based on a weighted average of the peak flows computed using 31 years of record at SCVWD #1, 43 years of record at USGS #1721, and 19 years of record for the 1976 Regional Equations. Resultant Regional adjustment is a decrease of 26% for 1% peak flows and a decrease of 31% for 10% peak flows.
4. SCVWD Urban Hydrology Method used downstream of USGS Station #1721.
5. All flows completely contained within the channel.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flow)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Upper Penitencia Cr. @ USGS Station #1721	3	21.10	4,300	6,100	1,300	1,800	1,800	560
Upper Penitencia Cr. @ SCVWD Station #1 (@ Gridley Street)	4	22.19	4,500	6,300	1,500	2,000	2,000	640
Upper Penitencia Cr. U/S Coyote Cr.	5	23.91	4,600	6,500	1,700	2,300	2,200	760

JAN 1979



LEGEND

- 28" — Mean Annual Precipitation
- Watershed Boundary
- Subwatershed Boundary
- Creek
- ⓪ Pick-up Point
- △ Stream-flow Gaging Station

Santa Clara Valley Water District

Upper Penitencia Creek Drainage Area

JAN 1979

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: EAST
Major Watershed: BERRYESSA CREEK, LOWER PENITENCIA CREEK, AND TRIBUTARIES
Study Date: JUNE 1978
Assumptions:
1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 15-minute intervals.
3. Hydrographs for rural areas balanced using 1976 Regional Equations.
4. SCVWD Urban Hydrology Method used downstream of Piedmont Road and on all tributaries except Sweigert Creek, Los Coches Creek and Tularcitos Creek.
5. All flows completely contained within the channels.
6. All values calculated by G. S. Nolte and Associates using SCVWD methodology.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Berryessa Cr. @ Piedmont Road	1	4.51	1600	2200	700	910	540	240
Sweigert Cr. U/S Berryessa Creek	2	.49	190	260	110	140	60	30
Berryessa Cr. D/S Sweigert Creek	2	5.00	1800	2400	770	1000	600	270
Crosley Cr. U/S Berryessa Creek	3	.93	250	350	190	240	130	60
Berryessa Cr. D/S Crosley Creek	3	5.93	2000	2800	950	1200	730	320
Sierra Creek @ Piedmont Road	16	.82	190	270	140	190	100	40
Sierra Creek @ Hostetter Road	17	1.47	330	450	270	350	180	90
Sierra Creek U/S Berryessa Creek	4	1.77	410	570	320	420	220	110
Berryessa Creek D/S Sierra Creek	4	7.70	2400	3300	1200	1600	950	430
Berryessa Creek @ Cropley Road	4	7.99	2500	3400	1300	1700	990	460

JAN 1979

BERRYESSA CREEK, LOWER PENITENCIA CREEK, AND TRIBUTARIES (Continued)
TABLE OF DESIGN FLOOD FLOWS

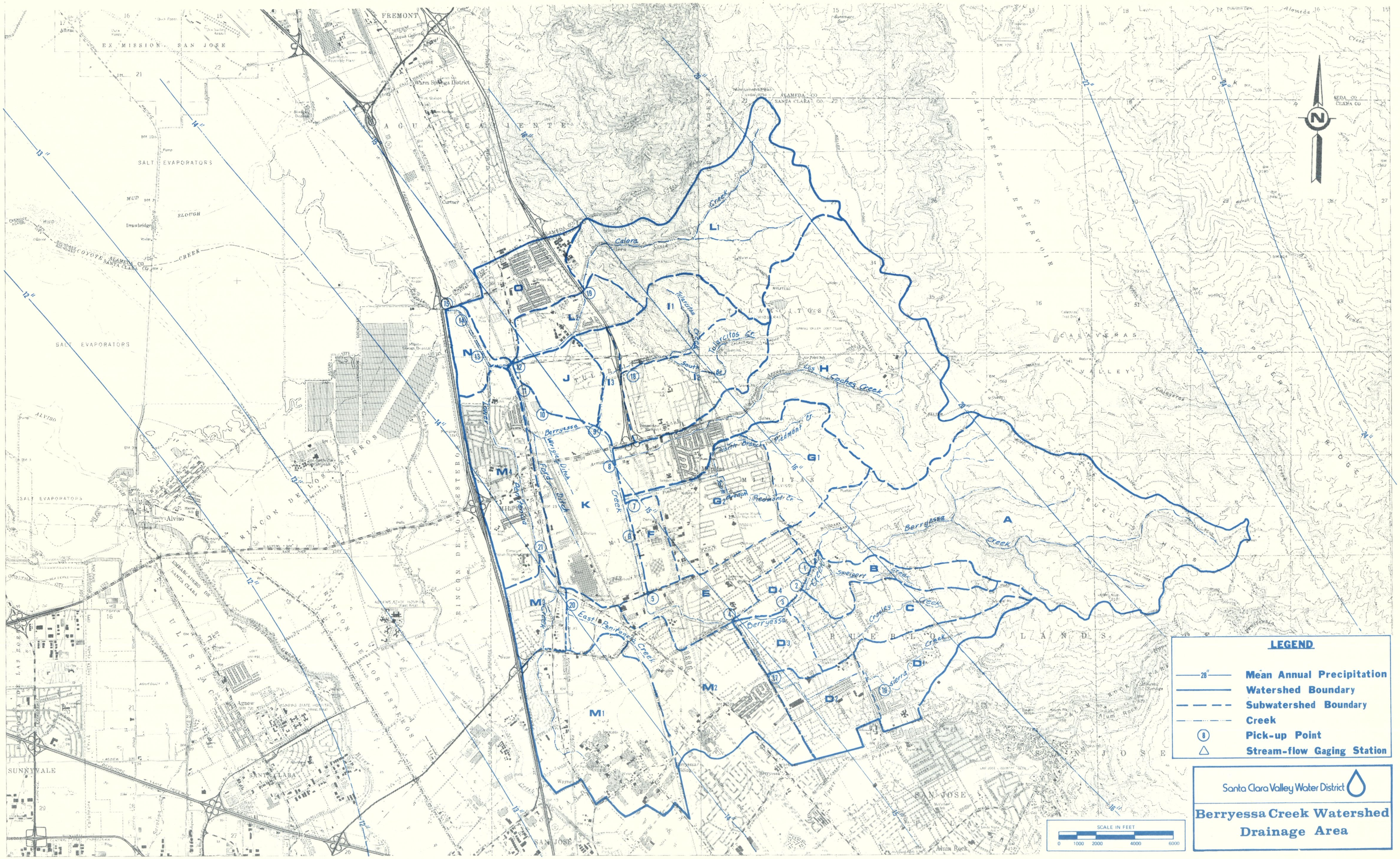
Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Berryessa Creek @ Landess Ave.	5	8.76	2600	3500	1400	1800	1100	520
Berryessa Creek U/S Piedmont Creek	7	9.12	2600	3500	1400	1800	1100	550
Piedmont Creek U/S Berryessa Creek	7	1.93	510	700	360	470	240	130
Berryessa Creek D/S Piedmont Creek	7	11.05	3000	4100	1700	2200	1400	680
Los Coches Creek U/S Berryessa Creek	8	4.02	1400	2000	620	800	470	200
Berryessa Cr. D/S Los Coches Creek	8	15.07	4000	5500	2300	3000	1800	880
Tularcitos Cr. U/S Berryessa Creek	9	1.96	450	620	340	440	230	120
Berryessa Creek D/S Tularcitos Creek	9	17.03	4500	6100	2600	3400	2100	1000
Berryessa D/S Drainage Lagoon	10	17.58	4500	6100	2600	3400	2100	1000
Ford & Wrigley Ditches U/S Berryessa	11	1.54	420	580	270	360	200	120
Berryessa D/S Ford & Wrigley Ditches	11	19.12	4900	6700	2900	3700	2300	1200
Calera Creek @ Highway 680	19	2.43	890	1200	410	530	280	130
Calera Creek U/S Berryessa Creek	12	2.93	920	1300	430	560	340	170
Berryessa Creek D/S Calera Creek	12	22.05	5600	7800	3200	4100	2700	1300
East Penitencia Creek @ Trimble Road	20	1.70	340	470	280	360	210	120
Lower Penitencia Creek @ Trimble Road	20	1.95	470	640	290	380	220	130
Lower Penitencia Cr. D/S East Penit.	20	3.65	810	1100	570	740	430	250
Lower Penitencia Cr. @ Curtis Ave.	21	3.99	890	1200	620	800	470	270

JAN 1979

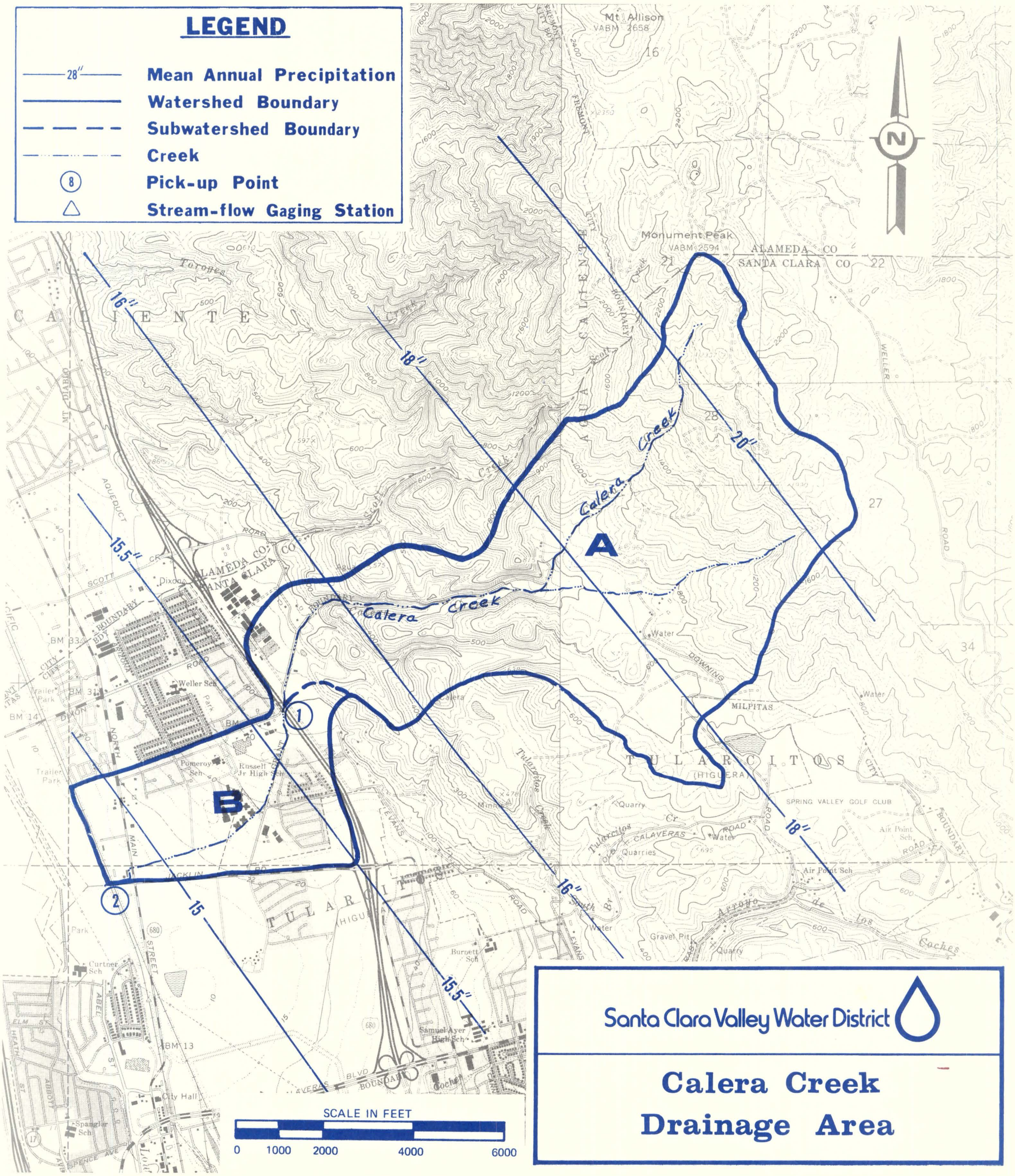
BERRYESSA CREEK, LOWER PENITENCIA CREEK, AND TRIBUTARIES (Continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Lower Penit. Cr. U/S Berryessa Creek	13	5.16	1200	1600	770	1010	610	350
Lower Penit. Cr. D/S Berryessa Creek	13	27.21	6700	9300	3900	5100	3300	1700
Sunny Hills Outfall U/S Lower Penit.	14	.81	240	330	170	220	110	60
Lower Penit. D/S Sunny Hills Outfall	14	28.02	7000	9600	4000	5200	3400	1700
Lower Penit. Cr. U/S Coyote Creek	15	28.26	7000	9600	4000	5300	3400	1700


Santa Clara Valley Water District

Berryessa Creek Watershed Drainage Area



Santa Clara Valley Water District

Calera Creek Drainage Area

JAN 1979

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: EAST

Major Watershed: LOWER SILVER CREEK AND TRIBUTARIES

Study Date: MARCH 1978

Assumptions:

1. 24-hour storm distributed using Army Corps of Engineers 1955 Storm Distribution for Hollister, Freedom and Stayton Mine (PL 18).
2. Hydrographs computed using HEC-1 with 15-minute intervals.
3. Hydrographs for rural areas balanced using 1976 Regional Equations as adjusted by 20% for 1% peak flows and 19% for 10% peak flows. Las Animas Adjustment is based on 14 years of record.
4. Santa Clara Valley Water District Urban Hydrology Method used downstream of Cribari Creek confluence and for lower reaches of all tributaries.
5. Peak 1% outflow from Lake Cunningham (3800 cfs) based on 1978 hydraulic analysis of Nolte's 1977 Lake design.
6. All flows completely contained within the channels.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Thompson Creek d/s San Felipe Creek	1	1.53	460	630	210	270	140	60
Thompson Creek u/s Cribari Creek	3	4.37	1200	1700	360	460	400	110
Cribari Creek u/s Thompson Creek	3	1.53	320	430	200	260	160	90
Thompson Creek d/s Cribari Creek	3	5.90	1500	2100	760	980	560	270
Thompson Creek u/s Yerba Buena Creek	5	6.33	1500	2000	800	1000	600	300
Yerba Buena Creek u/s Thompson Creek	5	2.58	530	720	320	410	250	140
Thompson Creek d/s Yerba Buena Creek	5	8.91	2000	2700	1100	1400	850	440
Thompson Creek u/s Evergreen Creek	7	9.37	2000	2700	1200	1500	900	480
Evergreen Creek u/s Thompson Creek	7	1.98	500	680	270	340	190	100
Thompson Creek d/s Evergreen Creek	7	11.35	2500	3400	1400	1800	1100	570

(Continued)

Santa Clara Valley Water District

DESIGN & CONSTR. UNIT
FD 301 (01-19-76)

MEMO ROUTING SLIP

No.	Name	Info	Action	Initial
1	J. Richardson		✓	JR
2				
3	W. Carlson			
4				
5				
6				
7				
8				

OK But please let your staff know the 78 letter is outdated.

Comments: Has someone asked for these. If not I suggest we wait until creeks are to be designed since Q's may change again. Also these changes shouldn't affect design

From: Bill C

Date: 14 Sept 79

B. Carlsen

DRAFT:AS:kj/9/11/79

Bill
Please review &
lets discuss
9/12

Mr. Ben White
MacKay and Somps
1955 The Alameda
San Jose, California 95126

Dear Ben:

The following ~~is~~ are the updated design ~~of~~ flows on tributaries of Silver Creek:

Creek	2% Flow	1% Flow
Flint	450	540
Ruby	240	260
Quimby	380	430
Fowler	540	630
Evergreen	450	500
Yerba Buena	470	530

Please note that some values in the 2% column differ from values provided to you in our letter dated July 20, 1978. The 1% flow values did not change. Since the 1% flows are those used in the design of the channel plus freeboard, we don't expect any changes in the overall sizing of the channel.

~~Thank you~~.

Sincerely,

John L. Richardson
Head, Project Development Branch
Design and Construction

c.c. B. Carlsen
A. Saah

LOWER SILVER CREEK AND TRIBUTARIES (Continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Thompson Creek u/s Fowler Creek	9	11.90	2500	3400	1500	1900	1100	620
Fowler Creek u/s Thompson Creek	9	2.78	590	800	320	410	240	130
Thompson Creek d/s Fowler Creek	9	14.68	3100	4200	1800	2300	1400	740
Thompson Creek u/s Quimby Creek	11	15.13	3100	4300	1800	2300	1400	780
Quimby Creek u/s Thompson Creek	11	2.17	430	590	270	350	210	120
Thompson Creek d/s Quimby Creek	11	17.30	3500	4800	2100	2700	1600	890
Thompson Creek u/s Lower Silver Creek	13	17.99	3600	4900	2100	2800	1700	940
Norwood Creek u/s Lower Silver Creek	13	2.18	440	600	280	360	220	130
Lower Silver Creek u/s Thompson Creek	13	1.88	200	270	200	260	200	140
Lower Silver Creek d/s Thompson Creek	13	22.05	4100	5600	2600	3300	2100	1200
Ruby Creek at White Road	15	1.55	260	350	180	240	150	100
Flint Creek at White Road	17	1.98	520	710	270	350	190	70
Lake Cunningham Inflow	18	26.23	5100	6900	3200	4100	2600	1500
Lake Cunningham Outflow	18	26.23	3800	5200	2300	3000	2500	1400
Lower Silver Creek u/s So. Babb Cr.	20	27.11	3900	5300	2400	3100	2500	1500
So. Babb Cr. u/s Lower Silver Creek	22	4.00	920	1300	450	580	350	170
Lower Silver Cr. d/s So. Babb Cr.	22	31.11	4200	5700	2600	3300	2800	1700
No. Babb Cr. u/s Lower Silver Creek	23	2.57	270	360	270	340	270	170
Lower Silver Cr. d/s No. Babb Creek	23	33.68	4500	6100	2800	3600	3100	1800

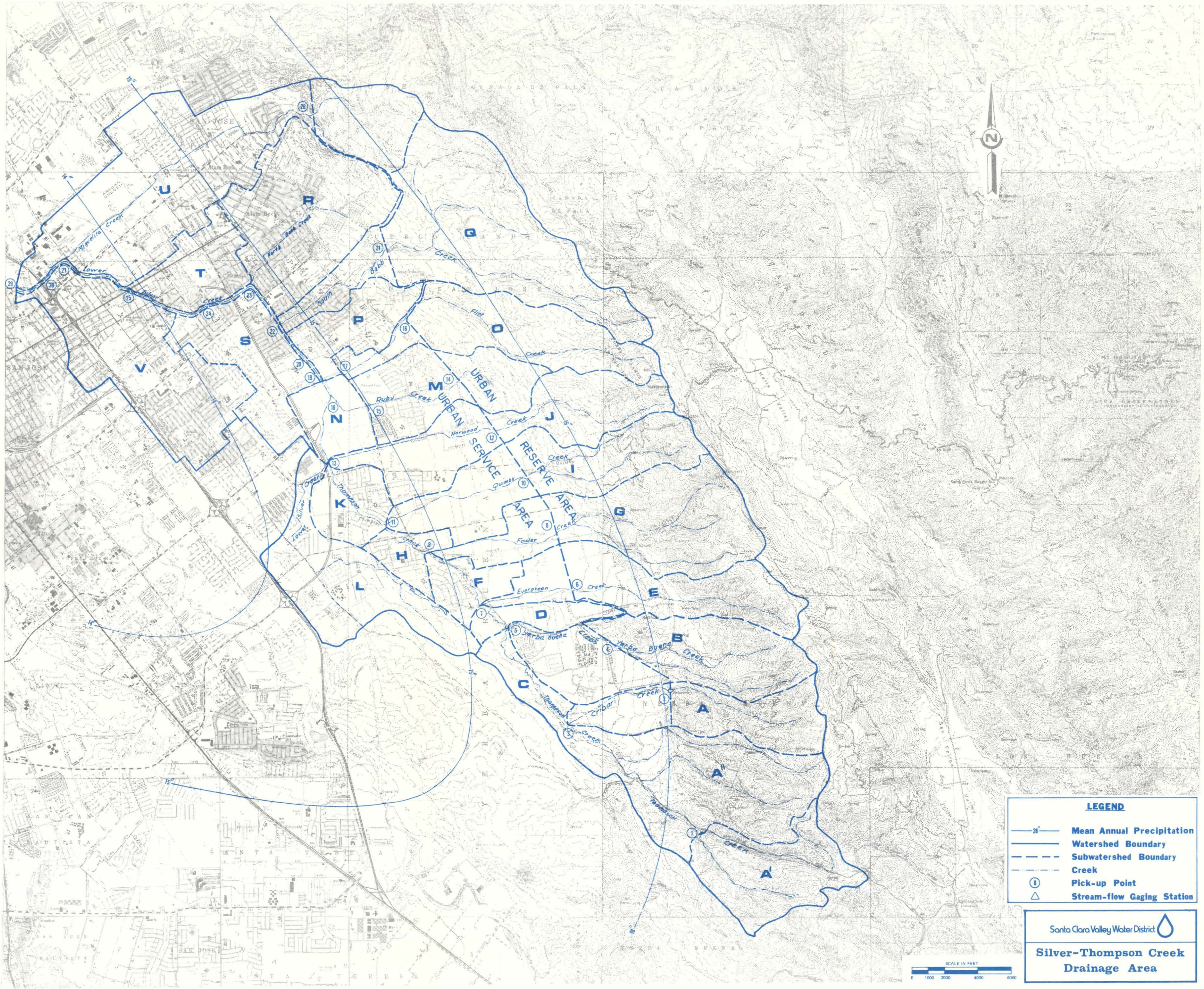
(Continued)

LOWER SILVER CREEK AND IBUTARIES (Continued)

TABLE OF DESIGN FLOOD FLOWS

			Peak Design Flows				24 Hr. Volumes (Average Flows	
			1%		10%			
Location	Catch Point	Drain Area (sq mi)	Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Lower Silver Creek at Jackson Avenue	24	35.19	4600	6300	3000	3800	3200	1900
Lower Silver Cr. at Alum Rock Avenue	25	36.15	4700	6400	3000	3900	3300	2000
Miguelita Cr. u/s Lower Silver Creek	27	4.49	600	810	480	620	450	440
Lower Silver Cr. d/s Miguelita Creek	27	40.64	5200	7200	3400	4400	3700	2200
Lower Silver Creek u/s Coyote Creek	28	43.50	5500	7600	3500	4500	4000	2300

JAN 1979



Santa Clara Valley Water District

Silver-Thompson Creek Drainage Area

JAN 1979

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: EAST
Major Watershed: COYOTE CREEK AND TRIBUTARIES
Study Date: SEPTEMBER 1977, CONFIRMED OCTOBER 25, 1977
Assumptions:

1. 72-hour storm distributed using Army Corps of Engineers 1955 storm distribution for Hollister, Freedom, and Stayton Mine (PL18).
2. Hydrographs computed using HEC-1 with one-hour intervals.
3. Hydrographs for areas upstream of Coyote Reservoir balanced using 1976 Regional Equations.
4. Hydrographs for rural areas downstream of Coyote balanced using 1976 Regional Equations as adjusted by 20% for 1% peak flows and 19% for 10% peak flows. Las Animas adjustment is based on 14 years of record.
5. SCVWD Urban Hydrology Method used downstream of Edenvale.
6. Coyote Reservoir coincidental initial storage = 16,000 acre-feet for 1% and 10% flows. Anderson Reservoir Corps of Engineers initial storage = 81,000 acre-feet for 1% flows. Coincidental initial storage = 76,000 acre-feet for 10% flows.
7. Downstream of Anderson 1% peak flows were made to match the Corps of Engineers 1977 study. The Districts values at these locations are within the 80% confidence interval.
8. All flows completely contained within the channels.

TABLE OF DESIGN FLOOD FLOWS

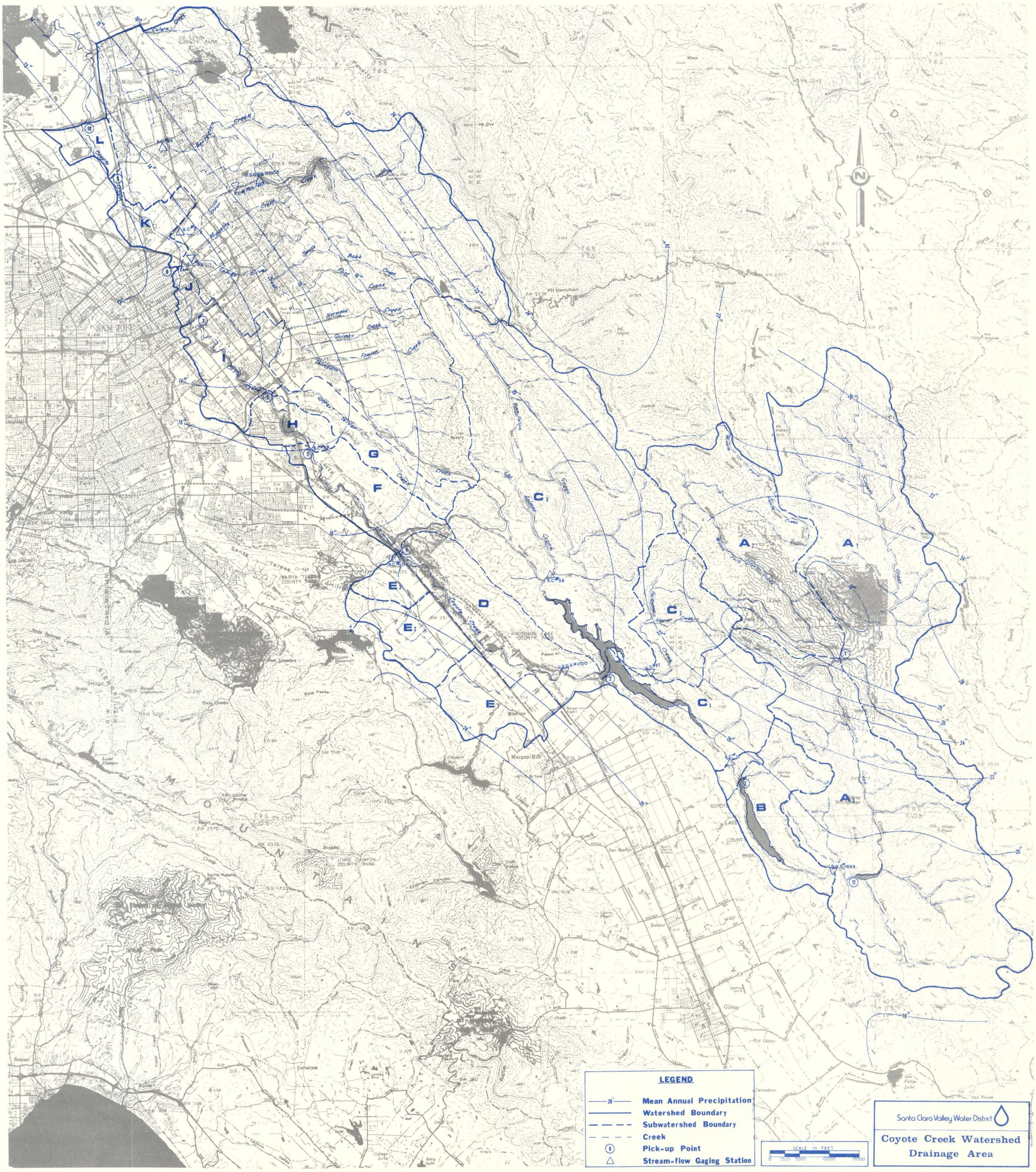
Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
East Fork Coyote Cr. U/S Middle Fork	1	41.00	10100	14100	4300	5700	4700	2100
Middle Fork Coyote Cr. U/S East Fork	1	22.00	6600	9200	2800	3700	2900	1300
East Fork Coyote Cr. D/S Middle Fork	1	63.00	16700	23400	7100	9400	7600	3400
Upper Coyote Cr. U/S East Fork	11	46.00	9900	13900	4000	5300	4600	1900
Upper Coyote Creek at USGS #1698	11	109.00	21800	30500	8600	11400	11400	4700
Inflow to Coyote Reservoir	2	118.68	21700	30300	9300	12300	12200	5100

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr, Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Outflow from Coyote Reservoir	2	118.68	13200	18500	3400	4500	9300	2400
Packwood Creek at SCVWD #57	31	10.10	3500	4700	1500	1900	1400	600
Inflow to Anderson Reservoir	3	192.72	20000	28000	7000	9200	15400	5000
Outflow from Anderson Reservoir	3	192.72	15000	21000	550	730	11600	550
Coyote Creek Upstream Fisher Creek	4	205.27	15000	21000	2300	3000	12100	1200
Upper Fisher Creek at Kalana Avenue	41	6.70	2100	2900	950	1200	770	360
Middle Fisher Creek at Bailey Street	42	6.68	2300	3200	980	1300	820	360
Fisher Creek Downstream Confluence	42	13.38	3800	5200	1700	2200	1600	720
Fisher Creek Upstream Coyote Creek	4	15.78	4000	5500	1900	2500	1900	850
Coyote Creek Downstream Fisher Creek	4	221.05	15000	21000	4100	5400	12700	2100
Coyote Creek at Edenvale(@ SCVWD #58)	5	229.05	15000	21000	4800	6300	12700	2300
Coyote Creek Upstream Silver Creek	6	232.62	14500	20300	3300	4400	12700	2400
Upper Silver Creek Upstream Coyote Cr.	6	5.96	950	1300	580	760	590	230
Coyote Creek Downstream Silver Creek	6	238.58	14500	20300	3600	4800	12900	2600
Coyote Creek at Highway 280	7	246.46	14500	20300	3800	5000	12900	2900
Coyote Creek Upstream Lower Silver Cr.	8	247.84	14500	20300	3700	4900	12800	2900
Lower Silver Creek Upstream Coyote Cr.	8	43.50	5500	7600	3500	4500	4000	2300
Coyote Creek D/S Lower Silver Creek	8	291.34	14500	20300	6800	7000		5200

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows 1% Peak (cfs)	Peak Design Flows 1% 80% Conf (cfs)	Peak Design Flows 10% Peak (cfs)	Peak Design Flows 10% 80% Conf (cfs)	24 Hr. Volumes (Average Flows) 1% (cfs)	24 Hr. Volumes (Average Flows) 10% (cfs)
Upper Penitencia Cr. U/S Coyote Creek	8	23.91	4600	6500	1700	2300	2200	760
Coyote Creek D/S Upper Penitencia Cr.	8	315.25	14500	20300	8200	10800		5900
Coyote Creek at Montague Expressway	9	319.32	14500	20300	8200	10800		6100
Coyote Creek at Highway 237	10	321.62	14500	20300	8300	11000		6100

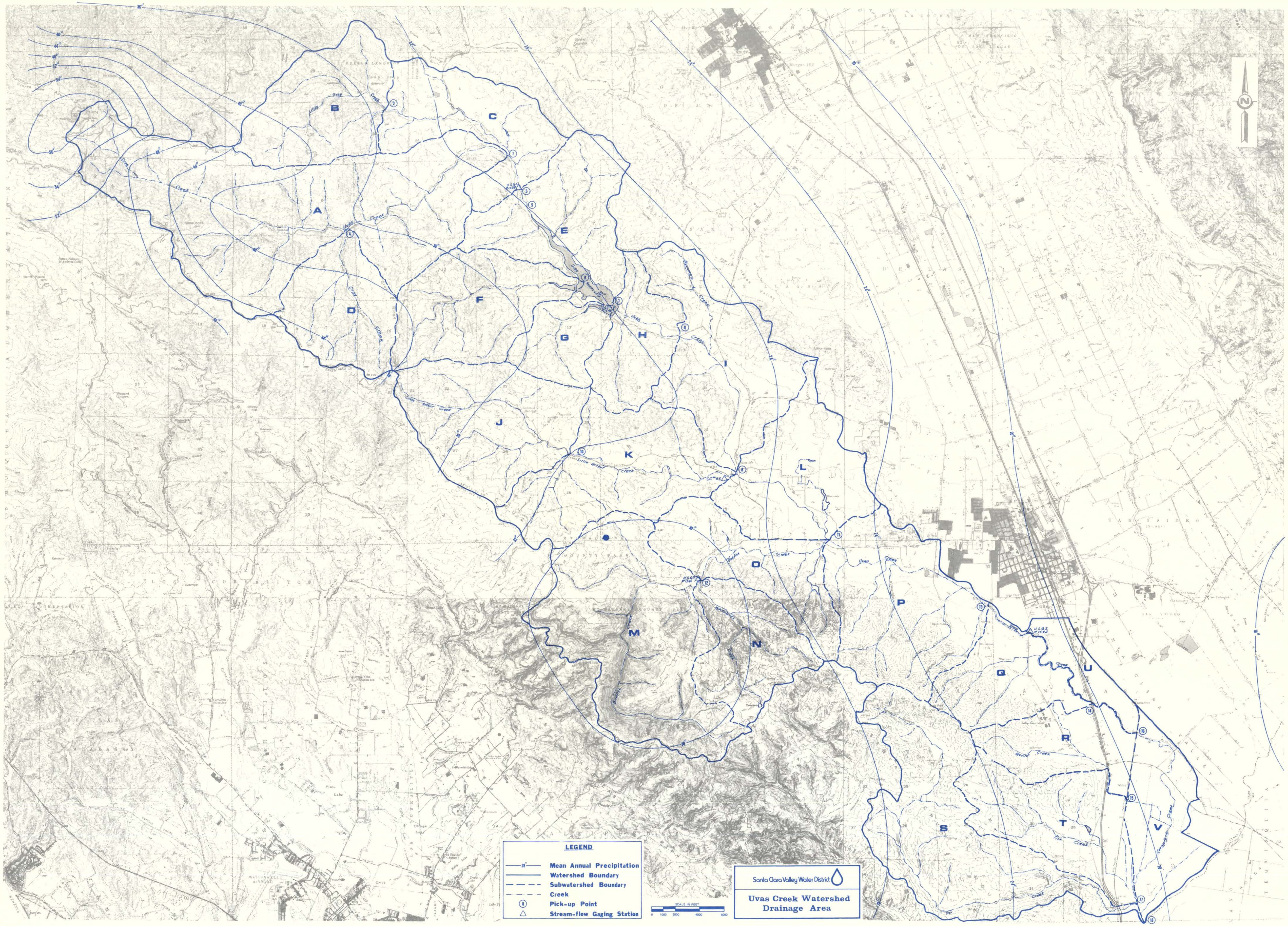


Santa Clara Valley Water District

Coyote Creek Watershed Drainage Area

JAN 1979

South



Santa Clara Valley Water District

Uvas Creek Watershed Drainage Area

JAN 1979

UVAS CREEK AND TRIBUTARIES (Continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Gavilan Creek @ SPRR	15	2.30	990	1300	460	590	320	150
Tar Cr. U/S Carnadero Creek	17	6.39	2700	3600	1200	1500	980	450
Tick Cr. U/S Carnadero Creek	17	3.55	1400	1900	640	820	480	220
Uvas-Carnadero Cr. U/S Pajaro River	18	87.50	16300	21000	8200	10100	10500	4800

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: SOUTH
Major Watershed: UVAS CREEK AND TRIBUTARIES
Study Date: MAY 1977
Assumptions:
1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 30-minute intervals.
3. Hydrographs balanced using 1976 Regional Equations.
4. Uvas Reservoir coincidental initial storage = 6800 AF for 1% flows and 6950 AF for 10% flows.
5. All flows contained within the channel.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Croy Creek U/S Uvas Creek	4	2.93	1400	1700	800	950	540	310
Uvas Creek U/S Little Uvas Cr.	1	14.19	5500	6900	3100	3700	2500	1400
Uvas Creek @ USGS #1539	3	21.35	7800	9700	4600	5500	3700	2000
Uvas Creek @ USGS #1540	7	30.46	11200	13900	6600	7900	5100	2800
Uvas Creek @ Watsonville Rd.	8	32.51	8100	10000	2600	3100	3700	1300
Uvas Creek U/S Little Arthur Cr.	9	37.25	8600	10700	2800	3400	4200	1600
Little Arthur Cr. @ SCVWD #65	9	9.46	3400	4200	1900	2300	1500	790
Uvas Cr. D/S Little Arthur Creek	9	46.71	10400	12900	3600	4300	5300	2300
Uvas Cr. U/S Bodfish Creek	11	50.34	10900	13600	4100	4900	5800	2500
Bodfish Creek @ USGS #1541	12	7.32	2500	3200	1300	1600	1000	520
Whitehurst Creek U/S Bodfish Creek	12	2.83	1200	1500	610	750	420	220
Bodfish Cr. U/S Uvas Creek	11	13.61	4300	5500	2500	3100	1900	1000
Uvas Cr. D/S Bodfish Cr.	11	63.95	13600	17200	6600	7900	7600	3500
Uvas Cr. D/S Reservoir Canyon Cr.	13	69.06	14200	17900	6800	8300	8200	3700
Uvas Cr. @ Highway 101	14	71.60	14400	18100	6800	8200	8500	3900
Uvas Creek @ SPRR	16	72.66	14500	18300	6900	8300	8600	3900

(Continued)

JAN 1979

Santa Clara Vall Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: SOUTH
Major Watershed: LLAGAS CREEK AND TRIBUTARIES
Study Date: AUGUST 1977
Assumptions:

1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 30-minute intervals.
3. Hydrographs for rural areas balanced using 1976 Regional Equations.
4. SCVWD Urban Hydrology Method used on West Little Llagas Creek, Princevale Drain, and Miller Slough, with existing storm drains.
5. **Existing Chesbro Reservoir coincidental initial storage = 6,000 AF for 1% flows and 6,400 AF for 10% flows.**
6. **All flows completely contained within the channels.**

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				**24-Hr. Volumes (Average Flows)**	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Inflow to Chesbro Reservoir	1	19.20	6,500	8,700	3,100	3,900	3,000	1,500
Llagas Creek D/S Chesbro Dam	1	19.20	3,900	5,100	800	1,000	1,800	550
Llagas Creek U/S Machado Creek	2	22.55	4,300	5,700	1,100	1,400	2,200	740
Machado Creek U/S Llagas Creek	2	1.34	650	850	340	420	210	110
Llagas Creek D/S Machado Creek	2	23.89	4,500	6,000	1,500	1,900	2,400	830
Llagas Creek U/S Hayes Creek	3	24.48	4,500	6,000	1,600	2,000	2,400	880
Llagas Creek U/S @ Monterey Hwy.	4	27.51	4,800	6,300	2,200	2,700	2,800	1,100
W. L. Llagas Cr. U/S Edmundson Cr.	5	5.03	1,600	2,100	1,000	1,300	680	380
Edmundson Cr. U/S W. L. Llagas Cr.	5	1.01	570	780	280	360	170	80
Llagas Cr. U/S E. L. Llagas Cr.	6	29.75	4,900	6,700	2,400	3,000	3,100	1,200
Madrone Channel U/S E. L. Llagas	10	5.09	2,100	2,900	860	1,100	700	300

(Continued)

JAN 1979

LLAGAS CREEK AND TRIBUTARIES XISTING CHESBRO (Continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Rucker Cr. U/S Llagas Cr.	14	2.24	1,100	1,500	490	630	310	140
Llagas Cr. D/S Rucker Cr.	14	60.53	12,400	17,000	6,900	8,900	7,000	3,100
Panther Cr. U/S Llagas Cr.	15	1.43	620	850	280	360	180	80
Llagas Cr. D/S Panther Cr.	15	61.96	12,600	17,300	7,000	9,100	7,200	3,200
Live Oak Cr. U/S Llagas Cr.	16	1.33	470	640	210	270	180	80
Llagas Cr. D/S Live Oak Cr.	16	63.83	12,800	17,500	7,200	9,300	7,400	3,300
Alamias Cr. U/S Llagas Cr.	17	3.24	1,200	1,700	500	650	380	160
Llagas Cr. D/S Alamias Cr.	17	67.07	13,300	18,200	7,400	9,600	7,700	3,500
W. Branch Llagas Cr. U/S Lions Cr.	20	5.91	2,500	3,400	1,100	1,400	880	390
Day Cr. U/S Lions Cr.	19	.93	500	660	240	300	140	70
Lions Cr. U/S W. Branch Llagas Cr.	20	3.82	1,500	2,000	700	870	530	250
W. Branch Llagas Cr. D/S Lions Cr.	20	10.66	3,600	4,900	1,900	2,400	1,500	700
Ronan Div. U/S W. Branch Llagas Cr.	21	13.53	1,400	1,900	610	790	470	200
W. Branch Llagas Cr. D/S Miller Slough	21	15.48	4,700	6,300	2,500	3,100	2,200	1,000
W. Branch Llagas Cr. U/S Llagas Cr.	18	17.74	5,100	6,800	2,700	3,300	2,500	1,100
Llagas Cr. D/S W. Branch Llagas Cr.	18	1.81	18,000	25,000	10,000	13,000	10,100	4,600

JAN 1979

(Continued)

LLAGAS CREEK AND TRIBUTARIES W, ISTING CHESBRO (Continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
E. L. Llagas U/S Corralitos Cr.	9	14.20	4,600	6,300	2,500	3,200	2,000	940
Tennant Cr. U/S Foothill Cr.	7	1.56	710	980	310	400	210	90
Foothill Cr. U/S Tennant Cr.	7	.43	250	330	120	150	60	30
Tennant Cr. D/S Foothill Cr.	7	1.99	860	1,200	410	530	270	120
Maple Cr. U/S Tennant Cr.	8	.63	340	470	150	190	90	40
Tennant Cr. D/S Maple Cr.	8	2.62	1,000	1,400	550	720	350	160
Corralitos Cr. U/S Tennant Cr.	8	1.27	560	770	250	320	160	70
Corralitos Cr. D/S Tennant Cr.	8	3.89	1,600	2,200	800	1,000	510	230
Corralitos Cr. U/S E. L. Llagas Cr.	9	5.63	2,200	3,000	1,100	1,400	760	330
E. L. Llagas Cr. D/S Corralitos Cr.	9	19.83	6,700	9,200	3,600	4,500	2,700	1,300
San Martin Cr. U/S Center Cr.	11	1.59	670	930	290	380	200	90
San Martin Cr. U/S New Cr.	12	2.64	1,000	1,400	530	690	330	150
San Martin Cr. U/S E. L. Llagas Cr.	13	3.74	1,400	1,900	790	1,000	480	220
E. L. Llagas Cr. D/S San Martin Cr.	13	24.49	7,400	10,200	4,100	5,300	3,300	1,500
Church Cr. U/S E. Llagas Cr.	6	1.75	810	1,100	340	440	230	100
Llagas Cr. D/S E. L. Llagas Cr.	6	56.92	12,300	16,800	6,800	8,700	6,500	2,900
Skillet Cr. U/S Rucker Cr.	14	1.31	640	880	280	360	180	80

(Continued)

JAN 1979

LLAGAS CREEK AND TRIBUTARIES XISTING CHESBRO (Continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Princevale Drain U/S Llagas Cr.	22	2.96	360	500	310	390	330	220
Llagas Cr. D/S Princevale Drain	22	87.77	18,400	25,400	10,300	13,100	10,500	4,800
Jones Cr. U.S Ortega Cr.	25	2.96	910	1,300	370	490	260	110
Ortega Cr. U/S Jones Cr.	25	2.58	970	1,400	400	530	290	120
Jones Cr. D/S Ortega Cr.	25	5.54	1,600	2,300	720	930	560	230
San Ysidro Cr. U/S Jones Cr.	26	4.26	1,600	2,300	640	830	510	210
Dexter Cr. U/S San Ysidro Cr.	26	1.70	770	1,080	320	420	220	90
San Ysidro Cr. D/S Dexter Cr.	26	11.50	3,700	5,200	1,600	2,100	1,300	530
Llagas Cr. U/s Pajaro River	24	103.65	21,100	29,000	11,400	14,500	12,300	5,500

Santa Clara Valley Water District
SUMMARY OF DESIGN FLOOD FLOWS

Zone: SOUTH
Major Watershed: LLAGAS CREEK AND TRIBUTARIES
Study Date: AUGUST 1977
Assumptions:

1. 24-hour storm distribution based on Army Corps of Engineers Standard Project Storm Distribution for Northern California.
2. Hydrographs computed using HEC-1 with 30-minute intervals.
3. Hydrographs for rural areas balanced using 1976 Regional Equations.
4. SCVWD Urban Hydrology Method used on West Little Llagas Creek, Miller Slough, and Princevalle Drain, with all planned storm drains and the W.L. Llagas and Ronan Diversions installed.
5. Enlarged Chesbro Reservoir, coincidental initial storage = 6,000 AF for 1% flows and 6,100 AF for 10% flows.
6. All flows completely contained within the channels.

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Inflow to Chesbro Reservoir	1	19.20	6,500	8,700	3,100	3,900	3,000	1,500
Llagas Creek D/S Chesbro Dam	1	19.20	1,900	2,500	290	360	1,300	220
Llagas Creek U/S Machado Creek	2	22.55	2,000	2,700	790	990	1,500	360
Machado Creek U/S Llagas Creek	2	1.34	650	850	340	420	210	110
Llagas Creek D/S Machado Creek	2	23.89	2,100	2,800	1,100	1,400	1,600	450
Llagas Creek U/S Hayes Creek	3	24.48	2,200	5,800	1,300	1,600	1,700	500
Hayes Creek U/S Llagas Creek	3	1.29	650	840	330	410	200	110
Llagas Creek D/S Hayes Creek	3	25.77	2,600	3,400	1,600	2,000	1,800	610
Llagas Creek U/S W. L. Llagas Cr.	4	26.73	2,900	3,800	1,800	2,300	1,900	690
W. L. Llagas Cr. U/S Edmundson Cr.	5	5.03	630	840	540	680	570	390
Edmundson Cr. U/S W. L. Llagas Cr.	5	1.01	570	780	280	360	170	80

(Continued)

JAN 1979

LLAGAS CREEK AND TRIBUTARIES W/ENLARGED CHESBRO (Continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24-Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Llagas Cr. D/S W. L. Llagas Cr.	4	33.82	4,500	6,100	2,800	3,500	2,700	1,200
Llagas Cr. U/S E. L. Llagas Cr.	6	36.06	5,000	6,900	3,000	3,800	3,000	1,400
Madrone Channel U/S E. L. Llagas	10	5.09	2,100	2,900	860	1,100	700	300
E. L. Llagas @ New Hwy. 101	9	2.80	1,200	1,600	530	690	390	170
E. L. Llagas U/S Corralitos Cr.	9	7.89	2,700	3,700	1,300	1,700	1,100	470
Tennant Cr. U/S Foothill Cr.	7	1.56	710	980	310	400	210	90
Foothill Cr. U/S Tennant Cr.	7	.43	250	330	120	150	60	30
Tennant Cr. D/S Foothill Creek	7	1.99	860	1,200	410	530	270	120
Maple Creek U/S Tennant Creek	8	.63	340	470	150	190	90	40
Tennant Creek D/S Maple Creek	8	2.62	1,000	1,400	550	720	350	160
Corralitos Cr. U/S Tennant Creek	8	1.27	560	770	250	320	160	70
Corralitos Cr. D/S Tennant Creek	8	3.89	1,600	2,200	800	1,000	510	230
Corralitos Cr. @ Middle Avenue	50	4.27	1,700	2,300	880	1,100	570	250
Corralitos Cr. U/S E. L. Llagas Cr.	9	5.63	2,200	3,000	1,100	1,400	760	330
E. L. Llagas Cr. D/S Corralitos Cr.	9	13.52	4,900	6,700	2,400	3,100	1,800	800
E. L. Llagas Cr. U/S San Martin Cr.	13	14.44	4,800	6,600	2,400	3,100	2,000	860
San Martin Cr. U/S Center Cr.	11	1.59	670	930	290	380	200	90
San Martin Cr. U/S New Cr.	12	2.64	1,000	1,400	530	690	330	150

(Continued)

LLAGAS CREEK AND TRIBUTARIES W/ [illegible]LARGED CHESBRO (Continued)

TABLE OF DESIGN FLOOD FLOWS

Location	Catch Point	Drain Area (sq mi)	Peak Design Flows 1% Peak (cfs)	Peak Design Flows 1% 80% Conf (cfs)	Peak Design Flows 10% Peak (cfs)	Peak Design Flows 10% 80% Conf (cfs)	24-Hr. Volumes (Average Flows) 1% (cfs)	24-Hr. Volumes (Average Flows) 10% (cfs)
San Martin Cr. U/S E. L. Llagas Cr.	13	3.74	1,400	1,900	790	1,000	480	220
E. L. Llagas Cr. D/S San Martin Cr.	13	18.18	5,700	7,800	3,000	3,900	2,400	1,100
E. L. Llagas @ Church Avenue	51	18.67	5,800	8,000	3,100	4,000	2,500	1,100
E.L. Llagas U/S Church Avenue	6	19.11	5,900	8,100	3,200	4,100	2,600	1,100
Church Creek U/S E. L. Llagas Cr.	6	1.75	810	1,100	340	440	230	100
E. L. Llagas Cr. D/S Church Cr.	6	20.86	6,400	8,800	3,500	4,500	2,800	1,200
Llagas Cr. D/S E. L. Llagas Cr.	6	56.92	11,200	15,000	6,300	8,100	5,600	2,600
Skillet Cr. U/S Rucker Cr.	14	1.31	640	880	280	360	180	80
Rucker Creek U/S Llagas Creek	14	2.24	1,100	1,500	490	630	310	140
Llagas Creek D/S Rucker Creek	14	60.53	11,500	15,400	6,500	8,400	6,100	2,800
Panther Cr. U/S Llagas Creek	15	1.43	620	850	280	360	180	80
Llagas Creek D/S Panther Creek	15	61.96	11,700	15,600	6,600	8,500	6,300	2,900
Live Oak Cr. U/S Llagas Creek	16	1.33	470	640	210	270	180	80
Llagas Creek D/S Live Oak Creek	16	63.83	12,000	17,000	6,700	8,700	6,500	3,000
Alamias Creek U/S Llagas Creek	17	3.24	1,200	1,700	500	650	380	160
Llagas Creek D/S Alamias Creek	17	67.07	12,600	17,300	7,000	9,000	6,900	3,200
W. Branch Llagas Cr. U/S Lions Cr.	20	5.91	2,500	3,400	1,100	1,400	880	390

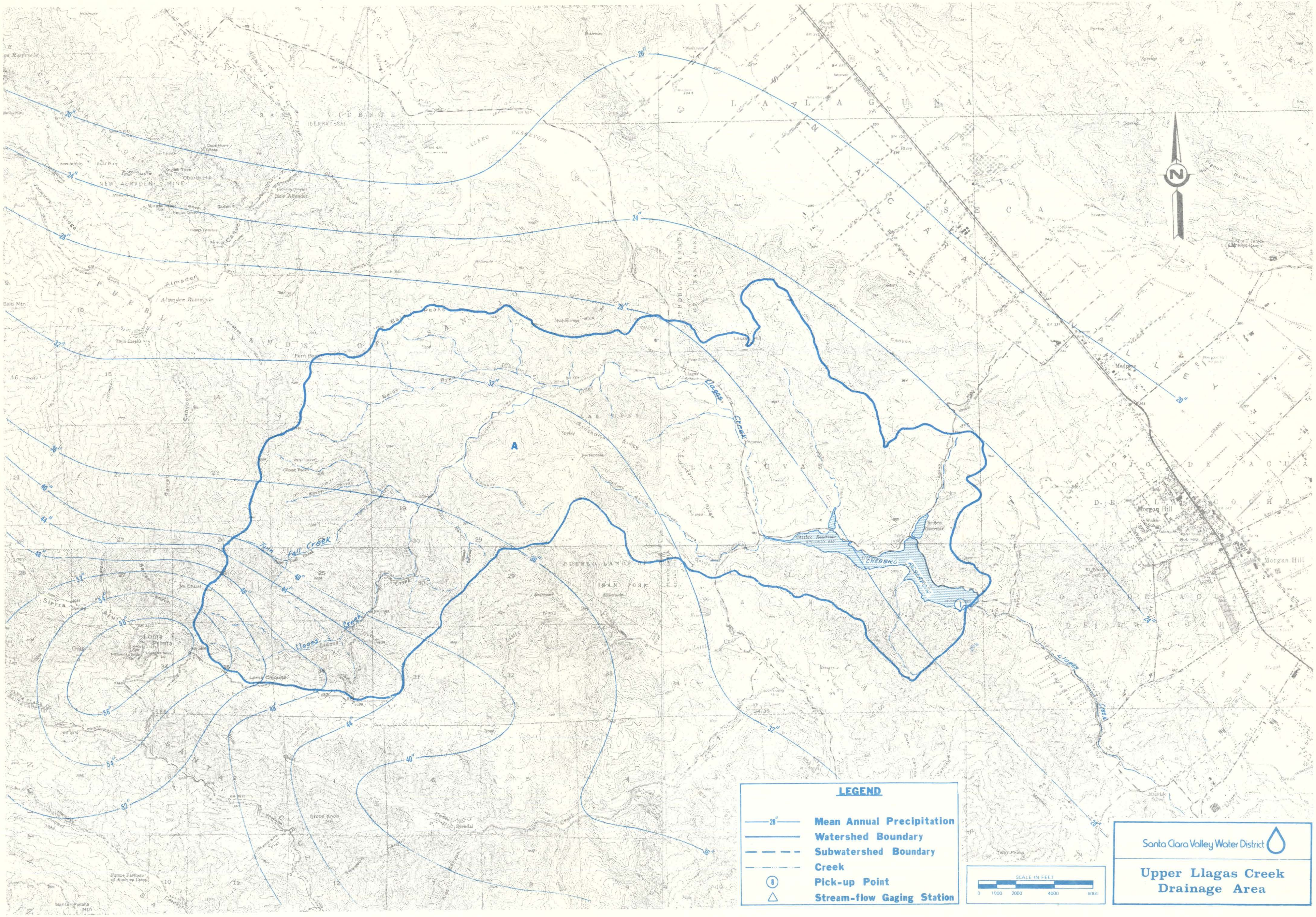
JAN 1979

(Continued)

LLAGAS CREEK AND TRIBUTARIES ENLARGED CHESBRO (Continued)

TABLE OF DESIGN FLOOD FLOWS

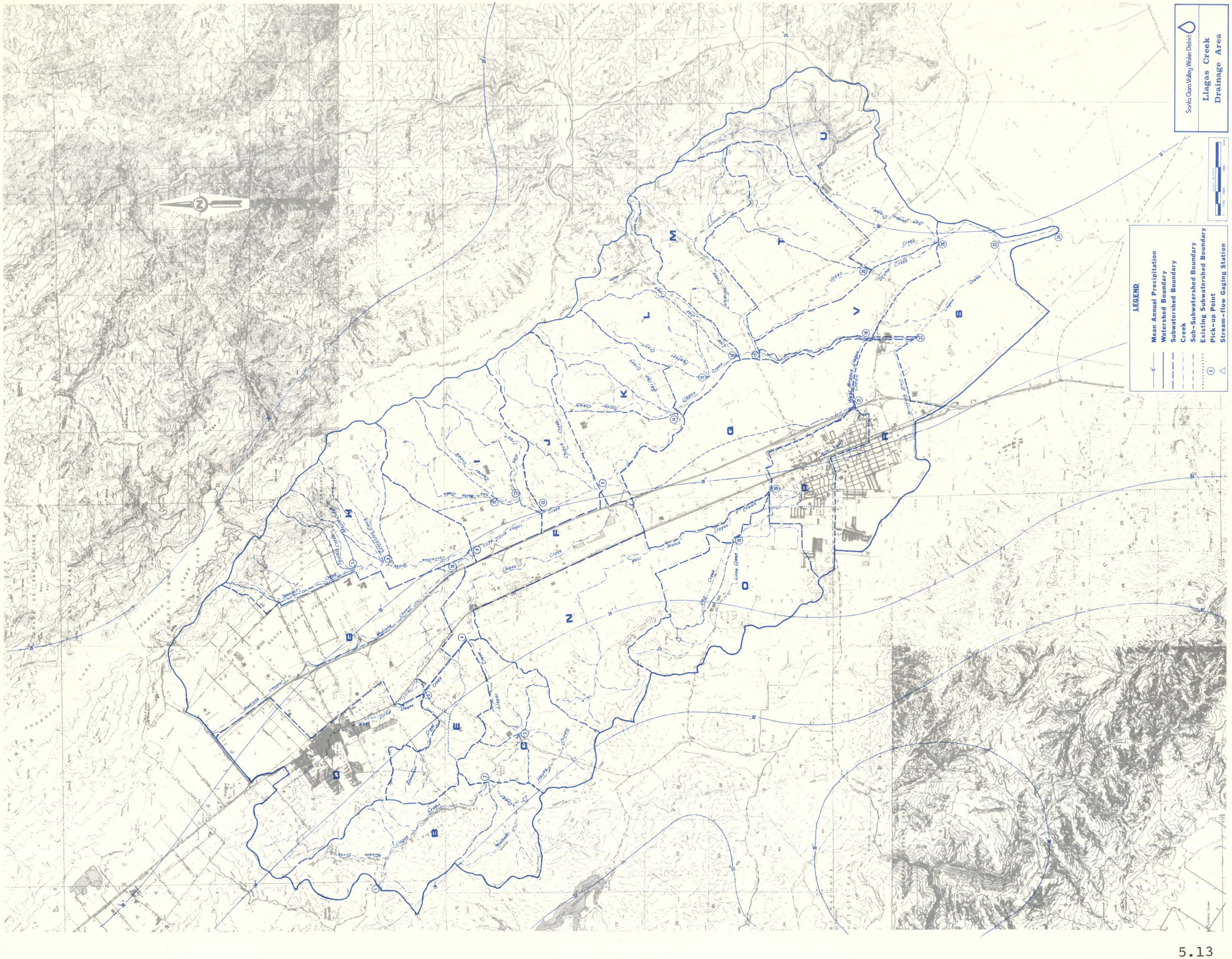
Location	Catch Point	Drain Area (sq mi)	Peak Design Flows				24 Hr. Volumes (Average Flows)	
			1%		10%			
			Peak (cfs)	80% Conf (cfs)	Peak (cfs)	80% Conf (cfs)	1% (cfs)	10% (cfs)
Day Creek U/S Lions Creek	19	.93	500	660	240	300	140	70
Lions Cr. U/S W. Branch Llagas Cr.	20	3.16	1,300	1,700	620	780	450	210
W. Branch Llagas Cr. D/S Lions Cr.	20	10.00	3,500	4,700	1,800	2,300	1,500	660
Ronan Div. U/S W. Branch Llagas Cr.	21	13.53	4,400	6,100	2,200	2,800	1,900	860
Miller Slough U/S W. Branch Llagas Creek	21	1.95	240	330	210	270	220	150
W. Branch Llagas Cr. D/S Miller Slough	21	15.48	4,600	6,200	2,400	3,100	2,100	1,000
W. Branch Llagas Cr. U/S Llagas Cr.	18	17.74	5,000	6,800	2,700	3,500	2,400	1,100
Llagas Cr. D/S W. Branch Llagas Cr.	18	84.81	17,100	23,500	9,500	12,300	9,300	4,300
Princevale Drain U/S Llagas Creek	22	2.96	360	500	310	390	330	220
Llagas Cr. D/S Princevale Drain	22	87.77	17,500	24,000	9,800	13,000	9,600	4,500
Jones Creek U/S Ortega Creek	25	2.96	910	1,300	370	490	260	110
Ortega Cr. U/S Jones Cr.	25	2.58	970	1,400	400	530	290	120
Jones Creek D/S Ortega Creek	25	5.54	1,600	2,300	720	930	560	230
San Ysidro Cr. U/S Jones Cr.	26	4.26	1,600	2,300	640	830	510	210
Dexter Cr. U/S San Ysidro Creek	26	1.70	770	1,080	320	420	220	90
San Ysidro Cr. D/S Dexter Cr.	26	11.50	3,700	5,200	1,600	2,100	1,300	530
Llagas Cr. U/S Pajaro River	24	103.65	20,300	28,000	11,000	14,000	11,400	5,200



Santa Clara Valley Water District

Upper Llagas Creek Drainage Area

NOV 1979



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Llagas Creek Drainage Area

JAN 1979